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CHEMICAL GROUTING

Progress Report of the Task Committee on Chemical Grouting of the Committee on Grouting of the Soil Mechanics and Foundations Division
(Proc. Paper 1426)

FOREWORD

In 1952, the ASCE appointed a Committee on Grouting with the aim of advancing the art and supplying information on grouting to the engineering profession. This Committee is divided into four Task Committees, one to cover each of the following grout materials: soil, cement, bitumen, and chemical. This report prepared by the Task Committee on Chemical Grouting summarizes the state of knowledge on this subject. The report is divided into the following main sections: Injection Processes, Injection Procedures, Summary of Field Applications, Patent Abstracts, and Annotated Bibliography.

During the construction and use of engineering structures the need often arises to alter some property of the material within or adjacent to the structures. One method of altering a property is to incorporate an additive either by mixing it with the material to be treated or by injecting (grouting) it into the in situ material. Troublesome zones which are readily accessible are usually treated by mixing (e.g. the blending of cement with soil to form a soil-cement road base); while inaccessible zones are often injected (e.g. the treatment of a pervious stratum one hundred feet below a dam.)

Injection is most commonly used to increase strength or decrease permeability; however, it can be used to alter a number of other properties. Strength decrease, permeability increase, compressibility decrease, frost susceptibility decrease, etc. also can be effected by injection.

Injection of grout is a very common technique of the oil industry. In drilling and operating wells, petroleum engineers frequently inject materials to reduce water flow or increase oil flow or perform some other function. While the civil engineer has made limited use of injection to aid construction in sandy soils and to help seal foundations, he does not exploit this valuable technique to the extent he could and probably should.

The main reasons the civil engineer fails to use injection fully and properly are his understandably scant knowledge of it and its usually high cost. Injection techniques are, unfortunately, relatively complex; the selection of proper grout and appropriate technique can normally best be made only after field exploration and testing (laboratory and/or field) have been conducted.

Note: Discussion open until April 1, 1958. Paper 1426 is part of the copyrighted Journal of the Soil Mechanics Division of the American Society of Civil Engineers, Vol. 83, No. SM 4, November, 1957.

By its nature, injection is a process whose results are often difficult to examine and to evaluate accurately. The art of chemical injection is largely a mystery to even the better informed engineers, primarily because of the large number of varieties and combinations of chemicals employed and the scarcity of practical data on the subject. While much has been written on chemical injection, most has been on sodium silicate and tends to be general in nature. As a matter of fact, some engineers think of "silicate injection" and "chemical injection" as synonymous, not realizing that there are many other chemical processes in common use (mostly by petroleum engineers).

Articles on chemical injection are sometimes written by people, not to inform the profession, but to promote their own interests. These articles usually avoid precise scientific descriptions, numerical data and stoichiometric equations. The profession can hope that comparative laboratory and field data on the various chemical injection processes will be forthcoming. The Task Committee on Chemical Grouting hopes that this report will show the potentialities and limitations of chemical grouting, and, thereby, promote increased use of this technique. The Committee will encourage laboratory and field experimentation aimed at learning more about new chemical grouts and developing better ones.

Respectfully submitted,

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I - INJECTION PROCESSES

In this section, the types of chemical reactions applicable to injection processes are described. For convenience, they have been classified into ten types as shown in Table 1; each type is discussed independently in the following paragraphs.

Dissolution

Certain constituents of soil or rock can be dissolved by the injection of appropriate substances; for example, calcium carbonate can be dissolved by hydrochloric acid, as $CaCO_3 + 2HCl \rightarrow H_2CO_3 + CaCl_2 \rightarrow H_2O + CO_2 + CaCl_2$ (water + carbon dioxide + calcium chloride, which is soluble). Dissolution can be used to increase permeability or to increase strength.

Dowell Inc. of Tulsa, Oklahoma has a regular service of "acidizing" water and oil wells to increase their rates of flow. They inject acid, alone or with additives, through the well into the water- or oil-bearing strata to dissolve soluble constituents, thereby increasing the permeability of the strata.

Potts (U. S. Pat. 1,635,500) proposes the injection of caustic soda into soil to increase its permeability.

Zemlin (U. S. Pat 1,820,722) suggests the injection of hydrofluoric acid into soil to dissolve some of the silica (SiO_2) and form silicon fluoride (SiF_4); the silicon fluoride acts on the earth salts and acids to set silica free again to cement soil particles, thereby increasing strength. This process could, in effect, form sandstone.

The M.I.T. Soil Stabilization Laboratory has conducted limited laboratory tests on the injection of organic soils to increase their permeability. Preliminary data show that hydrochloric acid can increase the permeability of an organic silt a hundredfold.

Ion Exchange

The nature of exchangeable ions can have a pronounced effect on the permeability of a fine-grained soil; for example, data on a montmorillonite at a given void ratio showed the calcium form to be 300 times as permeable as the potassium form (Cornell, 1951). While a dependable order of permeabilities cannot be given for the common ions, the sodium form of soil is one of the least permeable.

Since other soil properties, as well as permeability, depend on exchangeable ions, they can be altered by exchange reactions. For example, a sodium soil can be made less compressible by changing it to the calcium form.

The lagoon on Treasure Island, California, was made more watertight by filling it with sea water, thereby permitting sodium to replace the soil ions. Little practical use has, however, been made of ion exchange to improve the engineering properties of soil. The reasons for this limited use lie in the difficulty of injecting the relatively impermeable soils which are, unfortunately, the ones most affected by exchange reactions, and in the relatively modest alteration of soil properties that can be effected by ion exchange obtained through leaching with a simple electrolyte.

Soil Structure Alteration

The soil engineer has long known the great dependence of soil behavior on the relative positions of adjacent soil particles, or "structure." A clay may have its strength reduced from a high value to zero and its permeability reduced to 1/200 of its original value through an alteration of structure. While the most common manner of changing soil structure is mechanical, i.e. remolding, it can also be changed by leaching or injecting the soil with a chemical.

Lambe (1954) has described the nature and effect of soil-dispersant reactions. While ion exchange (usually the exchange of polyvalent ions with sodium ions) plays a part in the process, an increase in the electronegativity of the soil by polyanion adsorption is apparently a vital part of the treatment. Data have been presented to show that plasticity, density, compressibility, permeability, and frost susceptibility of fine-grained soil can be markedly altered by trace amounts of dispersants. The following data illustrate the effect of injecting a solution of sodium tetraphosphate (chemical weight equal to 0.1% of the dry soil weight) into two compacted soils.

Effect of Dispersant Injection

	Permeability in cm/sec	
	Before Injection	After Injection
Jamaica sandy clay	2.0×10^{-7}	1.5×10^{-8}
Pennsylvania silty sand	5.1×10^{-7}	2.2×10^{-8}

While dispersant injection is effective with only fine-grained soils and the results are modest in magnitude in comparison with other injection processes, it can be a very low cost treatment. A cubic foot of soil can be dispersant-treated at a cost for chemicals of 1 cent or less.

In addition to chemical dispersants which will reduce soil structure, there are chemical aggregants, which will increase soil structure by aggregating the small particles. Thus a soil can be made more porous and, thereby, more permeable by aggregant treatment. "Soil conditioners" are chemical aggregants used to make soil more porous.

Aggregant injection has less promise than dispersant injection for two reasons. While some soil aggregation can be effected at a constant soil volume, considerable aggregation usually requires a volume increase. This increase can be obtained with surface soils by mechanically fluffing them; a volume increase of a deeply buried soil would require movement against the overburden as well as against the injection seepage forces. Unlike the dispersants, the aggregants are relatively expensive chemicals.

Cooling of Thermoplastic or Molten Materials

Molten or hot thermoplastic substances can flow into porous substances and solidify upon cooling to permanently plug the voids. Soils can be made stronger and more impermeable by this process. Among the substances which have been suggested are: melted gypsum (U. S. Pat. 829,644), naphthalene vapor (U. S. Pat. 1,379,657), hot asphalt (U. S. Pat. 1,763,219), pitch and sulphur (U. S. Pat. 1,858,952), sulphur (U. S. Pats. 2,232,898 and 2,235,695),

metal alloys (U. S. Pats. 2,267,683 and 2,298,129) and resin (Canadian Pat. 386,475).

Pore Water Freezing

The pore water in a soil can be frozen by the injection of a refrigerant such as ammonia or carbon dioxide. This thermal solidification requires the maintenance of freezing temperatures and is, therefore, used only for temporary soil treatment.

Metathetical Precipitation

As one would logically expect, the most common injection process employs the introduction of chemicals in fluid form to react in the soil to produce an insoluble material. By plugging soil pores, the insoluble material reduces the permeability of the soil. Some products bind soil particles together and thus increase the strength of the soil. The two types of reactions which form insoluble products from solutions are metathetical precipitation, discussed in this section, and polymerization, discussed in the next section.

Table 2 summarizes many of the metathetical precipitation reactions that have been proposed. As a convenience to the engineer, the reactions have been divided into those which do not involve a soluble silicate and into those which employ a soluble silicate, primarily sodium silicate. Each group has been subdivided into those requiring single and those requiring multiple injections; all of the multiple injections are two-shot ones except for one which is three-shot.

Most of the processes listed in Table 2 were developed for use in treating oil wells. A few intended for special oil well problems may be of limited interest to the soil engineer; the great majority of the processes in Table 2 have been proposed for problems of interest to the civil engineer. The almost complete lack of published experimental and field data prevents even the beginning of a comparison of the many reactions. A single injection treatment has, of course, an obvious and major advantage over a multiple-shot injection, namely: the extent to which the separate solutions mix is never known. Also, the zone of mixing may in some cases be limited to the end of the injection pipe.

The civil engineer is more aware of silicate injection than he is of nonsilicate injection, although, oddly enough, the chemistry of the silicates is among the least known. The basic ingredient of nearly all of the silicate processes is a solution of sodium silicate in water, known as "water glass." This solution contains both free sodium hydroxide and colloidal silicic acid. The addition of salts or acids can cause the solution of salts or acids to form a gel, for example,

$$Na_{(2n-4)}(SiO_n) + HCl \rightarrow SiO_n(OH)_{4-2n} + NaCl$$

Very little is known about the structure of silica gel. It gives very little or no pattern when exposed to X-rays and is, therefore, considered amorphous. It has a high specific surface (600 sq. meters per gram; of the order of magnitude of montmorillonite) and is a very powerful and selective adsorbent. On aging, the gel shrinks (syneresis), becomes opalescent and cracks; it readily dissolves in caustic. The life of silicate stabilization when

the treated soil is exposed to air or to basic ground water may be limited.

Polymers

As pointed out in the preceding section, soil pores can be plugged with an insoluble product from a polymerization reaction. Polymerization is a reaction in which single organic molecules, called monomers, combine to form an organic macromolecule, called a polymer; it can be either an addition or a condensation combination. Nearly all resins are polymers and the terms are often used interchangeably.

Table 3 summarizes some of the injection processes employing polymers. Some of the processes inject polymers, others monomers and still others partially polymerized monomers. Heat, pressure, and catalysts are the three means used to trigger polymerization reactions; catalyst-initiated reactions are the most common and convenient for injection.

The in situ polymerization of injected monomer systems has a desirable feature which most of the other processes, both chemical and nonchemical, do not possess, namely: the ability to penetrate easily and uniformly all but the finest grained soils.

Synthetic polymers have a relatively high cost per pound. However, some of them can be used in solutions of very low concentration, making the cost of chemical per volume of soil treated as cheap as or cheaper than the low unit cost chemicals.

Emulsion Breaking

An emulsion is a dispersion of minute droplets of a liquid in another liquid in which it is not miscible. A viscous liquid, like a bitumen, can be dispersed in a nonviscous liquid, like water, to result in a relatively nonviscous fluid. Such dispersions can be injected into soils, and then broken (i.e. cause separation of the two components). This process leaves the soil pores filled with the viscous liquid.

Table 4 lists seven injection processes that involve emulsion breaking. Several of those in Table 4 employ bitumen; there are undoubtedly other bitumen emulsion injections. While bitumen processes should really be included under bituminous injection rather than chemical injection, the few in Table 4 are included for illustration of the emulsion-breaking process.

Suspension Separation

Table 5 lists processes which involve the injection of a suspension of solid matter in liquid followed by separation of the suspension. While suspension separation alone can be used as an injection process, it is usually employed with some other reaction. As a matter of fact, the injection of normal Portland cement grouts is an example of suspension separation since much of the suspension is separated in the ground prior to the set of the cement.

Particle Solvation

Certain substances will, in the presence of liquid, solvate and increase in volume. If such substances (usually in colloidal form) can be temporarily protected from contact with the liquid, they can be injected into soil pores where they later solvate and expand to plug the pores. Three U. S. patents (2,197,843; 2,300,325; and 2,329,148) on the use of this process for injection

are held by van Leeuwen. He has described systems which solvate and expand in water. The latter include:

1. Clay coated with an extract of kerosene treated with liquid sulfur dioxide.
2. Casein coated with certain oils.

The rate of solvation and swelling can be controlled by altering the pH of the system and polarity of the substances.

Sullivan (U. S. Pat. 2,320,954) employs particle solvation in a different manner. First a solution of sodium polyphosphate is injected and then a clay-water suspension is injected. The reaction really involved in this process is that described under Soil Structure Alteration; the injected clay is transformed into the sodium form, which then swells. In the light of our understanding of clay technology, we can see that the clay used should be a swelling type (e.g. montmorillonoid) containing exchangeable ions (Ca, Fe, etc.) which make it relatively nonswelling.

Table 1

INJECTION PROCESSES

Number	Type of Process	Example	Reference
1	Dissolution	Acid to Dissolve Silica	U. S. Pat. 1,820,722
2	Ion Exchange	Na^+ for Ca^{++} to Reduce Permeability	
3	Soil Structure Alteration	Dispersant Solution to Reduce Permeability	(Lambe, 1954)
4	Cooling of Thermoplastic or Molten Materials	Cooling of molten sulphur	U. S. Pat. 829,664 U. S. Pat. 1,379,657 U. S. Pat. 1,763,219 U. S. Pat. 1,858,952 U. S. Pat. 2,232,898 U. S. Pat. 2,235,695 U. S. Pat. 2,267,683 U. S. Pat. 2,298,129 Canadian Pat. 386,475
5	Pore Water Freezing	Freeze with CO_2	
6	Metathetical Precipitation	Sodium Silicate + Magnesium Chloride	See Table 2
7	Polymers	Injection of acrylate monomers to form water-sensitive polymer	Barker & Becker (1953)
8	Emulsion Breaking	Pine wood resin in alkali	U. S. Pat. 2,323,929
9	Suspension Separation	Silica in water	U. S. Pat. 1,430,306
10	Particle Hydration	Clay with protective coat	U. S. Pat. 2,329,148

Table 2

METATHETICAL PRECIPITATION

	Number	Process	Inventor	Patent Date	Patent Number
I. NON SOLUBLE-SILICATE					
A. Single Injection					
	1	Sodium Carbonate (Na_2CO_3) or Sodium Sulphate (Na_2SO_4) or water glass to mix with salt water in formations	Mills	1922	1,421,706
	2	Silicon tetrachloride or titanium tetrachloride in organic solvent to form ppt. on hydrolysis	Kennedy & Lawton	1935	2,019,908
	3	Soap solution to mix with oil or salt water in ground	Cannon	1937	2,079,431
	4	Salt of Sb,As,Bi,Sn, or Fe, to form ppt. on hydrolysis	Kennedy	1939	2,146,480
	5	Alkali phosphate + water-soluble soap	Grebe	1939	2,152,307
	6	Water-soluble aluminate + water-soluble soap	Grebe	1939	2,152,308
	7	Pb salt solution into salt water	Dunn	1939	2,156,219
	8	Metal alcoholates which hydrolyze	Bent, Loomis & Lawton	1939	2,169,458
	9	Water-soluble alginate + alkali hydroxide + organic material of weak acid properties or alkali metal pyro- or meta-phosphate	Bycle, Freeland & Lawton	1940	2,211,688
	10	Nonaqueous fluid + ester of silicon	Bent & Loomis	1941	2,259,875
	11	Ester of silicon which hydrolyzes	Bent & Loomis	1941	2,265,962

Table 2 (Continued)

Number	Process	Inventor	Patent Date	Patent Number
I. NON SOLUBLE-SILICATE (Cont'd)				
A. Single Injection (Cont'd)				
12	$Zn(OC_2H_5)_2$ = Zinc ethylate, or $Al(OC_6H_5)_3$ = Aluminum phenate, or $SnCl_3OC_2H_5$ = Trichlorostannic ethylate with accelerator	Kennedy	1942	2,270,006
13	Acid organic-silicate sol in state of incipient gellation	Stone & Teplitz	1942	2,281,810
14	One of reactants in colloidal form with protective film to delay reaction	van Leeuwen	1943	2,319,020
15	Introduce electrolyte which forms ppt. with naturally occuring (or added) ion; then passing a direct electric current through solution	Grebe & Chamberlain	1943	2,321,138
16	Arsenates or phosphates with metal salts	Williams	1943	2,332,822
17	Acaroid resin in alcohol solution	Lawton	1944	2,348,484
18	Sodium acid abietate	Miller	1945	2,369,682
19	Alakli soap of a petroleum hydrocarbon insoluble pine wood resin	Miller	1945	2,377,639
20	Aqueous solution of sodium carboxy methyl cellulose + salt	Wagner	1948	2,439,833

Table 2 (Continued)

Number	Process	Inventor	Patent Date	Patent Number
I. NON SOLUBLE-SILICATE (Cont'd)				
B. Multiple Injection				
1	Silicic acid-containing substance followed by chlorine gas	Muller	1931	1,815,876
2	Silicic acid-containing substance followed by carbon dioxide gas	Joosten	1931	1,827,238
3	Mg salt followed by alkaline hydroxide	Dunn	1939	2,156,220
4	Soluble alginate followed by $CaCl_2$ or $Al(SO_4)_3$, or $FeCl_3$	Ball	1939	2,174,027
5	Salt of organic acid solution followed by metal salt solution	Chamberlain and Robinson	1941	2,238,930
6	Ammoniacal solution of casein followed by formaldehyde	Vonder Ahe & Zweifel	1944	2,338,217
7	Fragmented magnesium followed by corrosive solution (e.g. NaCl)	Chamberlain	1945	2,378,687
II. SOLUBLE SILICATE (Primarily Sodium Silicate)				
A. Single Injection				
1	$Na_2SiO_3 + MgCl_2$ or $CaCl_2$ or HCl or CaO	Mills	1922	1,421,706
2	Unstable silicate gel + HCl or acid salts	Joosten	1937	2,081,541
3	Soluble silicate + water-soluble soap	Grebe	1937	2,090,626

Table 2 (Continued)

II. SOLUBLE SILICATE (Primarily Sodium Silicate) (Cont'd)

A. Single Injection (Cont'd)

Number	Process	Inventor	Patent Date	Patent Number
4	Unstable silicate gel, set controlled by dilution or pH	Vail	1938	2,131,338
5	Alkali silicate + Na bicarbonate or Na tetraborate or Na bisulphite	Malmberg	1939	2,176,266
6	Silicate + salt, then hydrolyzed by electric current	Reimers	1940	2,188,311
7	Na_2SiO_3 + HCl + carbon black	Lerch, Mathis, and Gatchell	1940	2,198,120
8	Water-soluble fluosilicate into salt water	Bent, Loomis, and Lawton	1940	2,200,710
9	Soluble silicate + base or salt	Lawton	1940	2,208,766
10	Sodium silicate + acid + coagulant	Langer	1941	2,227,653
11	Methyl silicate + HCl	Kennedy and Teplitz	1941	2,229,177
12	Na_2SiO_3 + HCl + Na bisulfate	Lerch, Mathis, and Gatchell	1941	2,236,147
13	Na_2SiO_3 + H_2SO_4	Reimers	1943	2,330,145
14	Fluosilicate + alkali	de Bataafsche Pet. Mij	1939	French 849,712

Table 2 (Continued)

Number	Process	Inventor	Patent Date	Patent Number
II. SOLUBLE SILICATE (Cont'd)				
B. Multiple Injection (Cont'd)				
1	$NaSiO_3$ followed by $CaCl_2 + CO_2$ (gas)	Jorgensen	1935	2,025,948
2	Na_2SiO_3 + bicarbonate salt followed by more bicarbonate salt solution	Anderson	1944	2,365,039
3	Material from $CaCl_2$ and $MgCl_2$, then material from NaOH and KOH, finally Na_2SiO_3	Hodgson	1948	2,437,387

Table 3

POLYMERS

Number	Materials	Inventor	Patent Date	Patent Number
1	Unsaturated fish oil + Petroleum distillate + Carbon tetrachloride + sulphur monochloride	Lerch, White, and Gatchell	1940	2,214,423
2	Styrene	Irons	1940	2,219,319
3	Vinylidene chloride or styrene	Maness	1940	2,219,325
4	Ester of a dicarboxylic acid and a polyhydric alcohol	Mathis	1941	2,252,271
5	Styrene or vinylidene chloride or phenol formaldehyde	Irons and Stoesser	1942	2,274,297
6	Thiourea + furfural (partial polymerization before injection)	Mathis and Rampacek	1943	2,307,843

Table 3 (Continued)

POLYMERS

Number	Materials	Inventor	Patent Date	Patent Number
7	Furfural + Urethane	Mathis and Rampacek	1943	2,321,761
8	Ammoniacal solution of casein followed by formaldehyde	Vonder Ahe and Zweifel	1944	2,338,217
9	Alkylated phenols or Styrene (partial polymerization before injection)	Buckley and Wrightsman	1944	2,338,799
10	Thiourea + Furfural	Lerch, Mathis, and Gatchell	1944	2,345,611
11	Thiourea + Furfural + fillers	Lerch, Mathis, and Gatchell	1944	2,349,181
12	Alkylated phenol-formaldehyde, or glycerol, etc.	Leverett and Wrightsman	1944	2,366,036
13	Partially condensed phenol-formaldehyde class resins	Kurtz and Sweely	1948	2,457,160
14	Partially condensed aldehyde + phenol and aldehyde + phenol + benzene	Cardwell	1949	2,485,527
15	Melamine-formaldehyde in gypsum cement slurry	Dailey	1949	2,492,212
16	Phenol-aldehyde + resorcinol-aldehyde + aldehyde	Seaver, Shepard and Less	1950	2,527,581
17	Formaldehyde + unsubstituted hydroxy aromatic compound	Wrightsman	1952	2,595,184
18	Acrylate of polyvalent metal, e.g., calcium acrylate	deMello, Hauser, and Lambe	1953	2,651,619

Table 4

EMULSION BREAKING

Number	Emulsion	Inventor	Patent Date	Patent Number
1	Bitumen + Soap + Casein in water	van Hulst	1936	2,051,505
2	Organic plastic medium	McKay	1937	2,071,758
3	Bitumen in water	Van Hulst	1937	2,075,244
4	Rubber latex into salt water	Irons	1938	2,121,036
5	Bitumen + filler (e.g., clay) in water	van Hulst and van Leeuwen	1936	2,158,025
6	Pine wood resin in alkali	Miller	1943	2,323,929
7	Tall oil + pine wood resin in water	Miller	1944	2,357,124

Table 5

SUSPENSION SEPARATION

Number	Suspension	Inventor	Patent Date	Patent Number
1	Silica in water	Francois	1922	1,430,306
2	Filler (clay, diatomaceous earth, etc.) in aqueous bitumen dispersion	McKay	1937	2,075,244
3	Filler (clay, iron, gelatin, etc.) in aqueous bitumen dispersion	van Hulst and van Leeuwen	1939	2,158,025
4	Cement + silica or fly ash in water	Wertz	1941	2,254,252
5	Cement + silica or fly ash in water	Wertz	1943	2,313,110
6	Lime + clay + iron oxide + cement	Mitchell, Marks, and Beene	1943	2,320,633
7	Barytes in water	Larsen	1946	2,393,173

II. INJECTION PROCEDURES

Introduction

The term "injection procedures" is taken to denote the physical aspects of the process of grouting, as distinguished from the chemical aspect that is the primary concern of Part I of this report. Included in the physical aspects of grouting are the location and drilling of grouting holes, setting of grouting pipes, mixing and injection of the grout, withdrawal of equipment, and cost of injection.

It is seldom on any grouting job that one can obtain sufficient information on the soil or rock conditions involved to assure that the grouting work will be successful. In rock grouting, for example, it is almost impossible to know in advance the degree of continuity between voids and cracks, even if these voids and cracks were originally found by core drilling at the site or by observation of seepage. In grouting unconsolidated sediments, it is likewise difficult to know whether pervious strata exist in continuous lenses, or whether such pervious strata are simply pockets unconnected with other pervious strata.

A carefully established plan of operation is, therefore, essential for each grouting job. This plan should be based on the fullest possible knowledge of soil or rock conditions, the greatest familiarity with the capabilities and limitations of the personnel and equipment available, and consideration of the type of chemical grout to be used. It should include a well developed layout of a pattern of holes, quantities of chemicals to be used, and timing of injections. Furthermore, the plan should provide for modifying the procedure during actual field operations if such should prove necessary.

Drilling of Holes and Setting of Pipes

After the purpose of the grouting operation has been well defined (such as improving strength of soil, reducing permeability, or reducing compressibility), and the location and number of grout holes determined, the grout holes must be drilled and prepared for treatment, or the grouting pipe must be driven in the proper manner to the proper depth.

A drilled and cased grout hole is typical for injection into rock, whereas a driven or jetted injection point is normally used in unconsolidated sediments.

For grouting rock, mechanical equipment to drill and prepare the hole for the grout pipe is necessary. Normally, rotary drilling is used; however, percussion drilling is satisfactory on certain projects. The drilling operation, especially percussion drilling, requires particular care in order that the cuttings will not lodge in and obstruct small fissures and pores in the rock. This fine material may hinder or prevent the flow of grout.

After the holes have been drilled into rock, the installation of casing in the hole is usually necessary to prevent caving or to isolate the zone to be treated. In some instances the casing is used as a pipe to carry the chemical grout to the zone to be treated. In other cases grout pipes are installed through the casing and into rock, using suitable packers to again isolate the

zone of treatment and to prevent backflow of the grout between the grout pipe and the casing.

On certain projects, the casing itself may be perforated by means of special explosive projectiles, and packers are placed above and below the perforated zone to isolate the treatment at that depth.

Where rock need not be penetrated, it is usually possible to drive or jet the grout pipe to the desired depth without the aid of any predrilling. The driving technique has the advantage that it results in a tighter contact between the pipe and the adjacent soil, thus minimizing the danger of piping action whereby the grout would rise to the surface along the pipe rather than penetrate the soil. During the jetting operation, as an alternate procedure, the water must rise to the surface, resulting in a flow channel adjacent to the pipe which may not close off after jetting has stopped, thus rendering it more difficult to penetrate the soil to be treated.

Plugging makes impossible the use of an open end pipe when the pipe is to be driven into the soil. Therefore, a section of perforated pipe with a suitable drive point or a commercial well point must be used which will stand up under the driving operation without becoming plugged.

Injection Operation

The equipment for mixing the chemicals depends on the chemical grout used. However, in general the mixing apparatus consists of supply tanks, mixing tanks, mechanical agitators, and suitable valves and connections.

Penetration of the material to be grouted is accomplished by pressure pumps, pneumatic pressure or gravity flow. A pulsating pressure as obtained from a positive displacement pump is usually more effective than a constant pressure grouting system.

Where pressure pumps are used, a suitable power source, pressure gauges, and connections are required. Where pneumatic pressure is used, an air compressor with sufficient pressure and volume capacity is required.

Figure 1 shows three typical layouts of grouting equipment. Figure 1a shows a schematic layout of equipment for injection of a so-called two-shot process, wherein first one chemical is pumped into the soil or rock, and then a second chemical is pumped through the pipe into the same stratum. Figure 1b is the layout for a one-shot chemical injection in which the two chemicals are mixed prior to injection. Figure 1c indicated a one-shot injection layout employing a single basic chemical.

The details and even the major features of a grouting layout differ widely from job to job, depending on the purpose of the work, the area to be grouted, the chemicals used, and many other factors previously mentioned. However, in general, the treated zone in the vicinity of injection points is limited in extent. Stage grouting techniques are often employed in which certain selected injection points are first grouted, and observations made in the field as to whether the grout rises in adjacent points, or whether the grout encounters refusal to penetration after a certain amount of grout has been pumped. If grout penetration is insufficient, it may be necessary to grout either to different depths or to use a closer spacing of injection points.

Even after the chemical grout has been injected in the desired quantities into the soil or rock, there is always a question, even as there is in cement grouting, as to whether the grouting material has "set" properly. If seepage

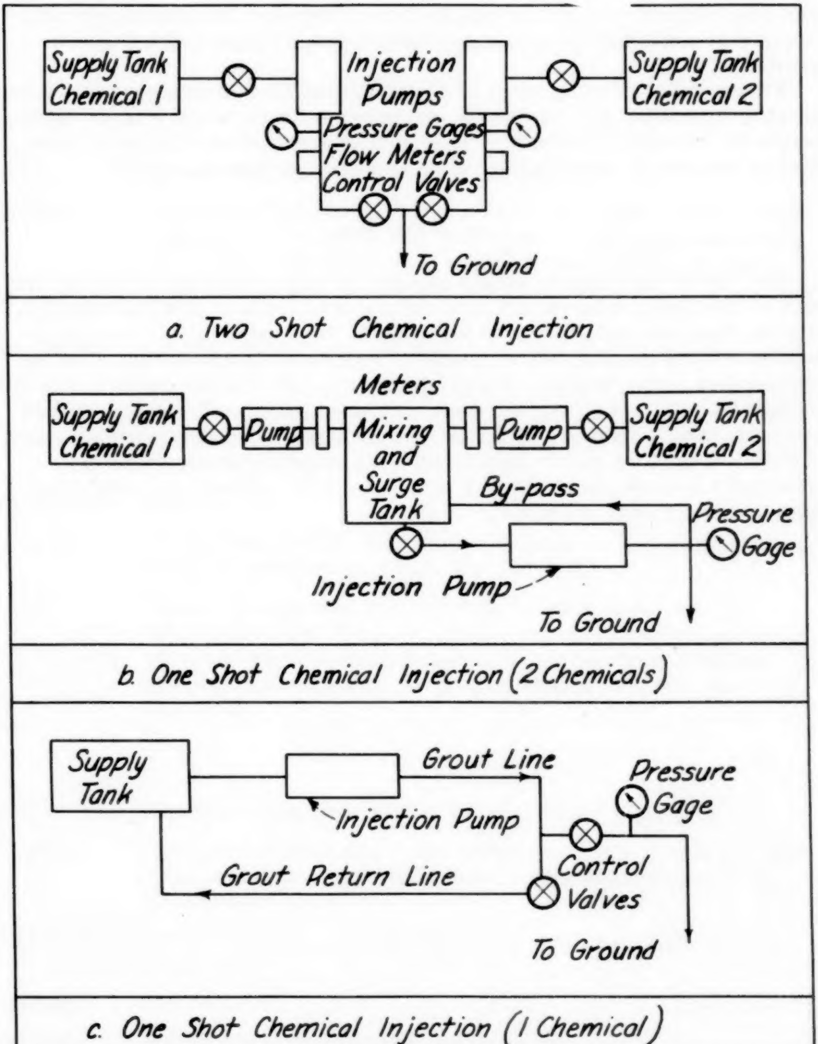

Fig. 1 - Typical Grouting Equipment Layouts

Fig. 1

existed prior to the grouting operation, the performance of the grouting program can be evaluated by observing the extent to which seepage has stopped. If seepage did not exist prior to the grouting, or if the purpose of the grouting was other than to control seepage, the performance of the grouting operation can only be evaluated by sampling or coring into the zone which has been grouted.

Withdrawal of grouting pipes and casing normally immediately follows the grouting operation, as it is usually necessary to clean out the pipe or casing before the chemical grout sets. This quick removal often requires the use of special equipment, and should be a major consideration on any job.

Cost of Injection

The cost of injecting chemicals into soil or rock depends on many factors such as the type and accessibility of the material treated, depth of injection, type of chemical, and the size of the project. Data available on the cost of sodium silicate grouting and acrylamide methylene bis acrylamide grouting indicate that on the majority of applications the cost of labor exceeds that of chemicals, with an approximate total cost range between $25.00 and $100.00 per cubic yard of treated soil. Thus, it may be seen that the present day cost of chemical grouting makes such procedures attractive to the engineer, principally in those cases where it is impossible to achieve the desired results by conventional methods.

III. SUMMARY OF FIELD APPLICATIONS

A. BRIDGE PIERS

(1)

Job:	Spree River, Berlin, Germany; bridge pier.
Purpose:	In enlarging the bridge it was necessary to sink a new caisson adjacent to and deeper than the old pier. The sand supporting the old pier was grouted to prevent it from sliding out from under the old pier during the sinking of the caisson for the new pier.
Chemicals:	Sodium silicate solution; sodium chloride solution (two-shot).
Methods:	Chemicals were injected through 16 holes drilled into the foundation under the old pier. Injection pipe was a 1-in. steel pipe, pointed at its lower end and perforated 20 in. from the point. The grouting was done in 20-in. lifts.
Results:	Satisfactory.
Reference:	"Solidifying Gravel, Sand and Weak Rock," Lars R. Jorgensen, Western Construction News, Nov. 10, 1931.
	"Neues Chemisches Verfahren zur Verfestigung des Baugrundes beim Erweiterungsbau der Eisenbahnbrücke uber die Spree am Bahnhof Jungfernheide der Berliner Ringbahn," J. Kuhnke, Zentralblatt der Bauverwaltung, Vol. 49, pp. 137-141, 1929.

(2)

Job:	Neuilly Bridge, near Paris, France; bridge pier.
Purpose:	Rather than carry one of the bridge piers to bedrock, the overlying sand was consolidated and the pier was founded on the consolidated mass.
Chemicals:	Sodium silicate, an acid, and heavy metal salt; KLM process (one-shot)
Methods:	Not reported.
Results:	Satisfactory; considerable saving of cost.
Reference:	"KLM - Chemical Grouting of Soils," advertising bulletin published by J. D. Lewin, 1937.

(3)

Job:	Bremen, Germany; piers of railway overpass.

A. BRIDGE PIERS (continued)

Purpose: Soil was solidified to increase the capacity of the bridge. The sand supporting the old piers was solidified to increase the bearing capacity and reduce settlement. Pier was rebuilt to withstand heavier loads.

Chemicals: Sodium silicate solution, I; salt solution not known, II; (two-shot).

Methods: Chemical I was injected through 3/4-in. pipes at 1 1/2-ft. depth intervals; Chemical II was injected at same interval but during withdrawal of pipes. Pipes were spaced 2 ft. apart; 360 pipes were driven to average depth of 15 ft. (11 to 32 ft.) A total of 350 cu. yds. of sand was solidified. Injection pressures ranged from 30 to 170 psi.

Results: The top layer (1 ft.) was weak (replaced by concrete). The solidified sand below this level was strong. A test section with 16 injection pipes (near pier foundation) was the subject of experimentation prior to commencement of the work.

Reference: "Baugrundverfestigung und Instandsetzung einer Eisenbahnbrücke unter Berucksichtigung des Schweren Kraftwagenverkehrs," W. Schröder, Bautechnik, p. 447, July 10, 1931.

(4)

Job: Berlin, Germany; piers of railroad bridge.

Purpose: To strengthen old piers, new concrete piles were to be constructed. To shorten the required length of the new piles and protect old piles, the sand between the old and new piling and under the new piles was solidified.

Chemicals: Joosten Process chemicals (two-shot).

Methods: The protective wall between piling 5 ft. thick and 40 ft. deep was solidified through three rows of injection pipes. No details of other procedures or equipment given.

Results: It was possible to shorten the required length of concrete piling about 12 ft. per pile. No difficulties were encountered in solidifying the sand.

Reference: "Pfeilerverstärkerung mit Nachträglicher Tiefgrundung der Reichsbahnbrücke über den Humboldthafen in Berlin," O. Mast, Bauingenieur, Vol. 15, pp. 327-331, Aug. 17, 1934.

B. BEACHES

(1)

Job: Not reported (military, USA); beach.

B. BEACHES (continued)

Purpose: For landing operations by armed forces on a beachhead, some type of roadway had to be provided. By chemical means and proper equipment an effective road was provided through originally loose beach sand.

Chemicals: Resinous binder (time of set controlled by extenders and catalysts).

Methods: A tractor-drawn "aniline-furfural;" binders are added in a pug mill and mixed directly; the sand is laid down and compacted in operations similar in idea to asphalt machine.

Results: Found to be efficient but very expensive. The sand can be stabilized at a rate of 17 ft. per minute for an 11-ft. roadway. The roadway will support heavy traffic in 10 to 14 hr.

Reference: "Current Navy Civil Engineering Research," Fred R. Kravath, The Military Engineer, p. 184, May-June, 1952.

"Beach Soil Stabilization; Aniline-Furfural Method," U. S. Naval Civil Engineering; Research and Evaluation Lab. Tech. Report R-001, Port Hueneme, Calif., 1950.

C. CANALS, CULVERTS, TRENCHES

(1)

Job: Sewer, Bay City, Michigan, U.S.A.

Purpose: Solidification of water-logged sand; sealing the bottom of a 9 1/2 ft. concrete sewer pipe. Protection and strengthening the roof of about 150 linear feet of an adjacent sewer tunnel against wash-out and cave-in.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Injection pipes driven vertically and at an angle to a depth of about 40 ft. below grade.

Results: Satisfactory.

Reference: Chemical Soil Solidification Co., Chicago, Ill. U.S.A.

(2)

Job: Sewer-Y and vault, Hammond, Indiana, USA.

Purpose: To solidify running sand around cofferdam for construction of a 6 ft. sewer-Y and vault about 20 ft. below grade.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Pipes were spotted at various angles to cover area around and below sewer after leaks at foot of sheet piling were stopped by vertical pipes set on 2 ft. centers.

Results: About 90-95 percent of leaks stopped sufficiently to pour concrete.

C. CANALS, CULVERTS, TRENCHES (continued)

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

(3)

Job: Sewer, Hammond, Indiana, USA.

Purpose: Solidification used to seal broken joints and holes in sewer caused by contamination of sewage water and by sulphuric acid contents in waste water of a nearby chemical plant.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Chemicals injected into ground through pipes. Material to be solidified was above ground water level.

Results: Leakages into sewer stopped.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

(4)

Job: Flood Sewer, Louisville, Kentucky, USA.

Purpose: Solidification used to facilitate mining through dry, running sand and to prevent sagging of existing 12" water main while excavating below.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Ground below water main solidified by chemical injection.

Results: Results satisfactory except that soil solidified too much, making excavation of ceiling below the water main cumbersome.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

(5)

Job: Canal at Agen, France.

Purpose: A ship canal parallels the Dordogne River. Undesirable variations in the water level in the canal were caused by seepage through a pervious stratum 18 feet thick.

Chemicals: Sodium silicate, acid, and heavy metal salt (one-shot).

Methods: Leakage was stopped by grouting a cut-off wall in the pervious stratum.

Results: Seepage reduced from 40 to 0.15 gpm per lineal foot.

Reference: "Grouting with Chemicals," Joseph D. Lewin, Engineering News Record, p. 61, Aug. 17, 1939.

(6)

Job: Railway culvert near Zackerick, Germany.

C. CANALS, CULVERTS, TRENCHES (continued)

Purpose: Chemical solidification used so that excavation for the culvert could be made in free running sand.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: The sides of the excavation were sealed with piling and the bottom was sealed with chemicals.

Results: Apparently satisfactory.

Reference: "The Development of the Joosten Process of Soil Consolidation During a Ten-Year Period of Practical Application," Dr. Eng. Adolph Mast, Bautechnik, Vol. 16, No. 21, Waterways Experiment Station, Translation No. 39-15, May 20, 1938.

(7)

Job: Adolph Hitler Canal, Germany.

Purpose: Chemicals used to seal leaks in sheet piling.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: No details.

Results: Apparently satisfactory.

Reference: "The Development of the Joosten Process of Soil Consolidation During a Ten-Year Period of Practical Application," Dr. Eng. Adolph Mast, Bautechnik, Vol. 16, No. 21, Waterways Experiment Station Translation, No. 39-15, May 20, 1938.

(8)

Job: Sewer-pipe trench in Hammerbrok, near Hamburg, Germany.

Purpose: To increase the bearing capacity of the sandy soil supporting the sewer pipe, the trench was dug to the watertable level, and the sand was solidified to a depth of 15 ft. below the water table.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Prior to chemical grouting, a concrete layer was placed over the bottom of the excavation, and injections were made through holes drilled through the concrete. The pipes were spaced 1 1/2 to 2 1/3 ft. apart. Pipes were driven by hand and a small high-pressure pump was used.

Results: Successful.

Reference: "Erfahrungen mit der Chemischen Bodenverfestigung und Anwendungsmöglichkeiten des Verfahrens," W. Sichardt, Bautechnik, pp. 181-186, Mar. 18, 1950.

C. CANALS, CULVERTS, TRENCHES (continued)

(9)

Job:	By-pass canal in London Docks, England.
Purpose:	A concrete rectangular by-pass canal subjected to high water pressures was leaking, especially in the joints of the concrete lining.
Chemicals:	Joosten Process chemicals (two-shot).
Methods:	Chemicals were injected into the lining of the canal. Equipment and procedures were not described.
Results:	Successful; from two photographs shown, one taken prior to and the other after the injection, it appears that the leakage of water was stopped.
Reference:	"Chemische Bodenverfestigung und Abdichtung," Karsten, Deutsche Bauzeitung, Vol. 69, pp. 476-478, 1935.

D. DAMS AND APPURTENANCE STRUCTURES

(1)

Job:	Genissiat Dam, France.
Purpose:	The upstream cofferdam leaked. The leakage occurred through 30-meter thick alluvial deposits of the Rhone River.
Chemicals:	Sodium silicate solution; hydrochloric acid; clay.
Methods:	Leakage was stopped by a threefold grouted curtain, composed of two curtains of clay between which was inserted chemical grouting.
Results:	Satisfactory.
Reference:	"The most Recent Precautions to Avoid the Formation of Pipings," A Mayer, Third Congress on Large Dams, Stockholm, Question No. 10, R22, 1948.

(2)

Job:	Ghrib Dam, Algiers.
Purpose:	Chemicals used in conjunction with cement in grouting the cut-off wall.
Chemicals:	Sodium silicate solution; aluminum sulfate.
Methods:	Holes were drilled about 6 ft. apart; about 9,200 tons of cement and 133 tons of chemicals were used.
Results:	Total leakage less than 0.1 cu. ft. per second.
Reference:	"Algerian Rockfill Dam Substructures," I. Gutman, Engineering News-Record, p. 749, May 26, 1938.

D. DAMS AND APPURTENANCE STRUCTURES (continued)

(3)

Job:	Bu-Hanifia Dam, Algiers.
Purpose:	At the abutments the foundations were badly faulted and required grouting. These foundations, consisting of finely fissured shale, could not be reached by the cut-off wall, and required grouting.
Chemicals:	Sodium silicate; acid and a heavy metal salt (KLM) (one-shot).
Methods:	Chemical grouting was used where cement grout would not penetrate; 15,500 tons of silicate used.
Results:	Satisfactory.
Reference:	"Algerian Rockfill Dam Substructures," I. Gutman, Engineering News Record, p. 749, May 26, 1938.

(4)

Job:	Camarossa Dam, Spain.
Purpose:	The limestone comprising the foundation leaked 390 sec. ft. when the dam was first put in use.
Chemicals:	Sodium silicate and probably a chloride (two-shot).
Methods:	Not reported.
Results:	Although job not completed due to lack of funds, leakage dropped to 93 sec. ft.
Reference:	"Solidification of Sand, Gravel, and Granular Materials by Chemical Means," advertising bulletin by Lars Jorgensen.

(5)

Job:	Lock and Dam No. 2, Mississippi River, USA.
Purpose:	The large building pit of a new lock being built next to an old one was protected against inflow of water through an existing row of old sheet piling by solidification.
Chemicals:	Joosten Process chemicals (two-shot).
Methods:	Injection pipes were driven through mud packed layers of fine sand; spacing of pipes about 24 in. c-c.
Results:	Leakage prevented.
Reference:	Chemical Soil Solidification Co., Chicago, Ill., USA.

(6)

Job:	Lock and Dam No. 3, Mississippi River, USA.
Purpose:	Chemical consolidation was considered as a means of stabilizing the foundation (silty muck).

D. DAMS AND APPURTENANCE STRUCTURES (continued)

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: Not reported.

Results: Chemical solidification was not attempted since laboratory tests indicated solutions would not penetrate.

Reference: Final Report: Laboratory Tests for Chemical Consolidation of Foundation Materials at Lock and Dam No. 3, Mississippi River, Corps of Engineers, 11-37.

(7)

Job: Anthony Falls Lock and Dam, Mississippi River, USA.

Purpose: Backfill filled trenches in St. Peter sandstone solidified to stop bypass of water into monolith pits.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Vertical injection pipes were placed along, and on both sides of sheet piling.

Results: Satisfactory.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

(8)

Job: Glen Canyon Dam site, Arizona, USA.

Purpose: It was considered desirable to attempt to waterproof and strengthen the porous sandstone foundation.

Chemicals: Sodium silicate and various reagents ($NaHCO_3$, $NaAlO_2$, $CaCl_2$) (one-shot).

Methods: Injection holes were drilled in initial test with relief holes 8 in. away.

Results: None of the solutions (including water) penetrated the formation at the location tested. Laboratory tests had indicated that some penetration could be obtained. Laboratory specimen was apparently not representative.

Reference: Bureau of Reclamation, Field Trip Report No. 745, by J. P. Elston, dated Nov. 10, 1949.

(9)

Job: Cofferdam near Kuttawa, Kentucky, USA.

Purpose: Due to a fault directly under the footing site and to the cofferdam being split by over-driving, water leaked in at a high rate. This water had to be eliminated before construction could be carried on. The fault was through solid rock but sand boils were encountered in the sides.

D. DAMS AND APPURTENANCE STRUCTURES (continued)

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: Exceedingly long injection pipes, up to 74 ft. in length, were used in the fault fissures; they were withdrawn as the chemicals were pumped in by two-solution method. To seal off the cofferdam, holes were burned in the sheet piling and "Y" pipes were welded in place. Chemicals were then pumped in.

Results: After pumping 11,412 gallons of sodium silicate and 8,000 gallons of calcium chloride into the site, the water was completely sealed off, bringing the work to successful completion. Cost $14,700.

Reference: "Chemicals Stop Cofferdam Leaks," C. Martin Riedel, Civil Engineering, Vol. 21:1-6, p. 195, Apr., 1951.

(10)

Job: Hydraulic Dam cores, Honolulu, T. H.

Purpose: Due to the permeable characteristics of the volcanic soil, it was impossible to keep water from seeping through. This acidic soil was treated both to consolidate it and to render it impermeable to water.

Chemicals: Alkali, 15 to 35 lbs. Na_2O per ton of soil.

Methods: Alkaline solution was mixed into the soil before compaction.

Results: Produced a 200 to 1000-percent more impermeable soil which flowed easily yet was readily compacted; it resembled a sticky clay.

Reference: "Chemical Treatment of Hydraulic Dam Cores," F. E. Hance, Engineering News-Record, Vol. 103, pp. 542-543, Oct. 3, 1929.

(11)

Job: Concrete conduit at Heart Butte Dam, North Dakota, USA.

Purpose: Stop water leakage into conduit through construction joints to prevent piping of semi-consolidated sandstone foundation, thereby eliminating potential settlement.

Chemicals: Lignin liquor, sodium dichromate, ferric chloride.

Methods: Holes were drilled through concrete in lower portion of conduit near seven leaking joints. Chrome-lignin grout was pumped into holes with a simplex grout pump until refusal under 90 psi pressure. Escape of grout through the leaks was controlled and finally stopped by caulking with lead wool. This permitted grout to circulate then "set" within the joint. Prior to injecting the chemical grout, raw lignin was pumped into the hole as a precaution against contamination of water by any toxic hexavalent chromium that might be leached from the grout gel, the raw lignin neutralizing the hexavalent chromium.

D. DAMS AND APPURTENANCE STRUCTURES (continued)

Results: All grouted joints were tight after 9 months' service.

Reference: "Field Experiences with Chemical Grouting," M. Polivka, L. P. Witte, and J. P. Gnaedinger. Paper presented at the Annual Meeting of the A.S.C.E. in New York City, N. Y., Oct., 1954.

(12)

Job: Earth-filled dam on Malapane River near Turawa, Germany.

Purpose: The sealing of portions of a steel cut-off wall under the dam was accomplished by chemical solidification for preventing water from seeping under the cut-off wall through a layer of sand.

Chemicals: Joosten Process chemicals (two-shot).

Methods: The location of the sand layers to be solidified was first found. To solidify the 14,000 sq. ft. of sand, injection pipes 1 1/4 in. in diameter were driven 1 1/2 ft. at a time and chemicals injected. Depth of layers solidified varied from 33 to 85 ft. Spacing of pipes was from 2 to 8 ft., depending on porosity of material.

Results: Successful.

Reference: "Der Staudamm des Staubeckens an der Malapane bei Turawa," Rosemann, Bautechnik, Jan. 3, 1936, pp. 3-6; Jan. 10, 1936, pp. 28-30; and Jan. 17, 1936.

"The Development of the Joosten Process of Soil Consolidation During a Ten-Year Period of Practical Application," Dr. Eng. Adolph Mast, Bautechnik, Vol. 16, No. 21. Waterways Experiment Station, Translation No. 39-15, May 20, 1938.

E. DRYDOCKS

(1)

Job: Graving dock in Long Beach, California, USA.

Purpose: During excavation, ocean water broke through under a sheet piling cut-off wall and flooded the excavation at the rate of 50 second ft.

Chemicals: Sodium silicate solution; sodium chloride solution (two-shot).

Methods: A chemically solidified cut-off wall was made to connect the bottom of the sheet piling with an impermeable clay stratum 9 ft. below.

Results: Leakage stopped.

Reference: "Solidification of Sand, Gravel, and Granular Materials by Chemical Means," Lars R. Jorgensen; an independently published advertising article, undated.

E. DRYDOCKS (continued)

(2)

Job: King George Drydock, in Southampton, England.

Purpose: Chemicals used to seal leaks.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: No details.

Results: Apparently satisfactory.

Reference: "The Development of the Joosten Process of Soil Consolidation During a Ten-Year Period of Practical Application," by Dr. Eng. Adolph Mast, Bautechnik, Vol. 16, No. 21, Waterways Experiment Station, Translation No. 39-15, May 20, 1938.

(3)

Job: Drydock VI, Kiel, Germany.

Purpose: During repairs and enlargement of the drydock, chemical solidification was employed in sealing off water which was seeping into the drydock along the joint below the old dock and the new addition. Another application of chemicals in this job was to seal off the water seeping through the concrete walls of the drydock.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Pipes were driven below concrete slab, and the two chemicals were injected to seal off joint. For walls, injection pipes were driven into and through the concrete walls, and chemicals were injected until the water stopped seeping through.

Results: The water was completely sealed off; eventually all walls of the drydock were watertight. After a few weeks, walls were dry. The salt sea-water had no effect on the chemical solidification process.

Reference: "Erfahrungen mit der Chemischen Bodenverfestigung und Anwendungsmöglichkeiten des Verfahrens," W. Sichardt. Bautechnik, pp. 181-186, Mar. 18, 1950.

F. FOUNDATIONS

(1)

Job: Large building foundation in Berlin, Germany.

Purpose: The building, constructed on soft ground, was slowly settling. The top strata (swampy soil) could not be consolidated by any other means.

Chemicals: Sodium silicate solution; sodium chloride solution (two-shot).

F. FOUNDATIONS (continued)

Methods: Concrete piles were driven through the mud stratum into quicksand, and around the bottom of the piles a large block of sand (6 ft. thick by 60 ft. long) was solidified.

Results: Satisfactory.

Reference: "Solidifying Gravel, Sand, and Weak Rock," Lars B. Jorgensen, Western Construction News, No. 10, 1931.

(2)

Job: Church foundation at Ribe, Jutland.

Purpose: Part of the foundation was slowly settling and cracking the walls of the church.

Chemicals: Sodium silicate solution; sodium chloride solution (two-shot).

Methods: Inclined grout pipes were driven in under the old foundation and the chemicals injected, thereby widening and deepening the foundation.

Results: Satisfactory.

Reference: "Solidifying Gravel, Sand, and Weak Rock," Lars B. Jorgensen, Western Construction News, No. 10, 1931.

(3)

Job: Pumping plant foundation, Buford Trenton Project, USA.

Purpose: Chemical consolidation was considered as a means of stabilizing the foundation (impermeable soil).

Chemicals: Sodium silicate solution; sodium and calcium chloride (two-shot).

Methods: Not reported.

Results: Chemical solidification was not attempted because laboratory tests indicated that solutions would not penetrate.

Reference: U. S. Bureau of Reclamation Laboratory Report No. P-26, Jan. 16, 1942.

(4)

Job: Davis Dam Project, USA; warehouse foundation.

Purpose: It was considered desirable to chemically stabilize and strengthen the warehouse foundation soil.

Chemicals: Sodium silicate solution; sodium and calcium chloride (two-shot).

Methods: A number of 4-in. auger holes were drilled in the bottom of the footing excavation, filled with pea gravel, and the chemicals successively poured in.

F. FOUNDATIONS (continued)

Results: Satisfactory.

Reference: U. S. Bureau of Reclamation Report, "Memorandum Report Chemical Solidification of Warehouse Foundation," B. C. Wilkas, Davis Dam Project, Oct. 2, 1947.

(5)

Job: Office building foundation, England.

Purpose: The ground under a foundation wall was solidified to permit excavating along side for the deeper foundation of an adjacent building.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: Not reported.

Results: Satisfactory.

Reference: "Underpinning and Foundation Work in Loose and Waterlogged Ground by Means of Chemical Consolidation, Ground Water Lowering and other means," H. J. B. Harding, The Structural Engineer, Vol. 14, Part 2, pp. 289-294, June, 1936.

(6)

Job: Sugar mill at Cleviston, Florida, USA; machinery foundation.

Purpose: Settlement of foundation under heavy machinery was stopped by injecting chemicals into the wet, medium-fine sand.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: Sand under the foundation solidified to a depth of 18 ft. Cost was $23 per cu. yd. of solidified soil.

Results: Satisfactory. However, it was concluded that clay, mud and silt are unsuitable for this treatment.

Reference: "Chemical Injection," Construction Methods, p. 78, Dec., 1945.

(7)

Job: Grodek Hydro-Electric Plant, West Prussia, Germany.

Purpose: (1) To grout the wingwalls which were of poor concrete, (2) to increase the bearing power of the sand below the foundation to prevent further settlement, (3) to fill cracks caused by uneven settling.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: Vertical holes, 43 ft., were drilled through the concrete of the wingwalls and into the ground below and grouted in stages of 6 1/2 ft. A total of 61 holes, 33 in concrete, were grouted. Average distance between them was 2 2/3 ft.

F. FOUNDATIONS (continued)

Results: Apparently satisfactory.

Reference: "Chemische Abdichtung von Bauwerken und Baugruben," W. Sichardt, Bautechnik, Vol. 11, pp. 455-457, 1933.

(8)

Job: Excavation for pumping station foundation at Bergdorf, near Hamburg, Germany.

Purpose: In the construction of a foundation for an underwater pumping station a retaining wall was required. The underlying wet sand flowed and gave rise to dangerous seepage when the shaft was drained. Chemical grouting eliminated the seepage and, in effect, extended the retaining wall to an underlying clay stratum.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: Not reported.

Results: Satisfactory.

Reference: "Chemische abdichtung von Bauwerken und Baugruben," W. Sichardt, Bautechnik, Vol. 11, pp. 455-457, 1933.

(9)

Job: Foundation pit at Berlin Siemenstadt, Germany.

Purpose: This was a field trial to determine if building excavation could be kept dry in waterlogged soil.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: Sheet piling was rammed to a depth of 28 ft. on three sides of the excavation site. Soil was removed down to the water table and solidification was then employed to seal the bottom and fourth side of the site. Dry excavation was then possible down to the upper surface of the petrified slab which was 5 ft. thick.

Results: The petrified wall withstood water at a pressure of 26 ft. of head. No leakage except a little through the sheet piling which was readily pumped out.

Reference: "The Chemical Solidification of Loose Soils," K. A. Pohl, Engineering Progress, Vol. 13, pp. 85-88, 1932.

"Die Anwendung des Chemischen Verfestigungsverfahrens bei der Abdichtung eines Langendammes und bei Schachtdichtungsarbeiten auf dem Kaliwerk Sachsen-Weimar in Unterbreizbach (Röhn)," Kali, Vol. 24, pp. 81-85.

(10)

Job: Postoffice foundation in Königsburg, Germany.

F. FOUNDATIONS (continued)

Purpose: Chemical consolidation used to reinforce foundation.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: The soil underneath the foundation was consolidated into blocks 15 ft. wide, 8 ft. deep, and extending the length of the building.

Results: Apparently satisfactory.

Reference: "The Development of the Joosten Process of Soil Consolidation During a Ten-Year Period of Practical Application," Dr. Eng. Adolph Mast, Bautechnik, Vol. 16, No. 21, Waterways Experiment Station Translation No. 39-15, May 20, 1938.

(11)

Job: Printing Works, machinery foundation, Germany.

Purpose: Chemical consolidation used to reinforce foundations and minimize vibration in heavy machinery.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: The soil was consolidated under building to a depth of 6 ft.

Results: Apparently satisfactory.

Reference: "The Development of the Joosten Process of Soil Consolidation During a Ten-Year Period of Practical Application," Dr. Eng. Adolph Mast, Bautechnik, Vol. 16, No. 21, Waterways Experiment Station Translation No. 39-15, May 20, 1938.

(12)

Job: Foundation piers for floodlights at the Kezar Stadium in San Francisco, California, USA.

Purpose: The soil being a loose or "free-running" sand, it was found that by chemical stabilization the excavation for foundation piers could be made without the use of shoring and the soil could be removed with hand tools.

Chemicals: Sodium silicate; hydrochloric acid and copper sulfate (one-shot) also sodium silicate and sodium bicarbonate (one-shot).

Methods: Injection needles spaced circumferentially 12 1/2 in. c-c about pier sites. The chemicals were pumped by Triplex positive-displacement pump under a maximum pressure of 200 psi. "One-solution" method was used.

Results: Soil left stabilized yet workable with hand tools. It results in a soil-compression strength of approximately 75 psi.; 500 gallons of solution were used for each pier.

F. FOUNDATIONS (continued)

Reference: "Running Sand Chemically Solidified," M. Polivka, Western Construction News, Vol. 24, pp. 81-53, July 15, 1949.

"Footings Dug Without Shoring in Chemically Solidified Sand," Thomas Bristow, Engineering News-Record, Vol. 143, p. 48, Nov. 10, 1949.

(13)

Job: Foundations for power transmission lines from Owens Gorge to Los Angeles through Mojave Desert, USA.

Purpose: The desert sands were unstable, making it difficult to place footings for tower. By stabilization, standard drilling augers were used. This method of soil stabilization prevented after-drilling cave-ins and sloughing; it also eliminated the necessity for concrete forms.

Chemicals: Sodium silicate solution; sodium bicarbonate solution (one-shot).

Methods: The solution was mixed in a tank, using first the sodium silicate and water, stirring well; the soda solution was then pumped in and the solution was immediately injected into the footing site. Reversing the procedure, by adding the silicate to the soda solution caused a premature gelling. Solution was injected by means of 12-ft. needle (1/2 and 3/4-in. galvanized pipe) using 100 psi. pressure from duplex piston.

Results: Very satisfactory. Effective in all sand areas. It gelled in 45 to 60 min. Increased dilution increased gelling time but reduced strength. Less water gave higher strengths.

Reference: "Soil Stabilization Speeds Tower Footing Excavations," E. D. Gershay, Electrical World, Jan. 15, 1951.

(14)

Job: Dwelling foundation at Spandau, Germany.

Purpose: To stabilize alluvial and clayey sand in order to save on lengths of foundation piles. The soil was incapable of supporting the building without piling, but the piles would have been of uneconomical length unless soil stabilization were employed.

Chemicals: Sodium silicate solution; salt solution (two-shot).

Methods: The piles were first driven down to the sandy layer; the sand was then solidified by the chemical injections, using the two-shot method.

Results: A 62-ft. long by 16.5-ft. wide and 5.5 to 6.5 ft. thick layer was produced. Approximately 1000 ft. of piling was saved, requiring about 50 piles, 30-ft. long, instead of an equal number of piles 55 ft. long.

F. FOUNDATIONS (continued)

Reference: "The Chemical Solidification of Loose Soils," K. A. Pohl, Engineering Progress, Vol. 13, pp. 85-88, 1932.

"Die Praktische Anwendung des Chemische Verfestigungsverfahrens von Losen Bodenarten bei der Grundung eines Wohnbauses," Deutsche Bauzeitung, Vol. 62, Suppl., Konstruktion und Ausführung, pp. 82-83, 1928.

(15)

Job: Pillar foundation at University Eye Hospital in Berlin, Germany.

Purpose: The sole of a bridge abutment was 42 ft. higher and in the direct vicinity of the pillar. It was feared that the abutment would be endangered during construction, and it was decided to create a solidified intermediate block between the bridge abutment and the pillar.

Chemicals: Joosten Process chemicals (two-shot).

Methods: The two-shot method was used to solidify a sort of buttress wall about 42 ft. deep and 13 ft. wide by 52 ft. long.

Results: The solidified-soil structure was successful in preserving the stability of the abutment. Strengths of 300 to 400 psi. were obtained. The work took only three weeks to complete.

Reference: "The Chemical Solidification of Loose Soils," K. A. Pohl, Engineering Progress, Vol. 13, pp. 85-88, 1932.

"Grundung der Neubauten für die Augen-und-Frauen-Klinik der Berliner Universität," A. Hansen, Zentralblatt der Bauverwaltung, Vol. 51, pp. 588-600, 1931.

(16)

Job: Brick foundation for a heavy hoist at a mine in Silesia, Germany.

Purpose: Due to cracks opening up in the brickwork (up to 3/8 in. wide), the whole foundation was shaking under the vibrations of the hoist.

Chemicals: Joosten Process chemicals (two-shot).

Methods: For the preliminary treatment, injection holes were drilled in various planes intersecting the cracks. Wider cracks were filled with sand and cement slurry, followed by the two chemicals of the Joosten Process. As a final treatment, the finer cracks were sealed with the chemicals.

Results: The preliminary treatment stopped the shaking, after which the finer cracks were sealed. No further disintegration of the foundation has occurred.

F. FOUNDATIONS (continued)

Reference: "Chemical Joint Sealing and Soil Solidification," Engineering News-Record, Vol. 127, pp. 222-225, Aug. 14, 1941.

(17)

Job: Foundation raft for a building extension at Kingston-on-Thames, England.

Purpose: It was necessary to put in a new foundation raft 6 ft. below water level. The work had to be carried out from the cellars of the existing building which was in full occupation. There was only 7 to 8 ft. of head room to work in.

Chemicals: Joosten Process chemicals (two-shot).

Methods: First the ballast was chemically solidified below all main walls and columns for a depth of 2 ft. below new excavation level. A shallow well-pumping system was installed, and water was pumped out until sufficient steel work of the new building had been erected to counterbalance the flotation of the water.

Results: Successful.

Reference: "Underpinning and Foundation Work in Loose and Waterlogged Ground by Chemical Consolidation, Ground Water Lowering, and Other Means," H. J. B. Harding, The Structural Engineer, Vol. 14, Part 2, pp. 289-294, June, 1936.

(18)

Job: Wall foundation, Akron, Ohio, USA.

Purpose: Solidification of medium sand under wall foundation of an office building. The foundation was badly settling causing a crack at the center of the wall.

Chemicals: Joosten Process chemicals (two-shot).

Methods: One row of injection pipes placed outside, and one row inside. 1-5/8 in. holes were drilled in floor slab on 3 ft. centers.

Results: No further settlement reported.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

(19)

Job: Foundation work on steel tanks, Eastern USA.

Purpose: Solidification to prevent dangerous settlement of 50 ft. high steel tanks.

Chemicals: Joosten Process chemicals (two-shot).

F. FOUNDATIONS (continued)

Methods:	Ring-like solidified area created under circular concrete footing slab and under square and independent concrete footings carrying roof columns.
Results:	Settlement stopped at some circumferential areas.
Reference:	Chemical Soil Solidification Co., Chicago, Ill., USA

(20)

Job:	Gang saw foundation, Longview, Washington, USA.
Purpose:	To stop settlement of heavily vibrating gang saw foundation in a timber plant near the Columbia River. Material to be solidified was fine to coarse sand below the ground water level.
Chemicals:	Joosten Process chemicals (two-shot).
Methods:	Vertical and 45^{o} injection pipes reached under heavy concrete foundation. Some holes were drilled through 7 ft. of concrete to reach subsoil core.
Results:	Satisfactory.
Reference:	Chemical Soil Solidification Co., Chicago, Ill., USA.

(21)

Job:	Foundation of elevator pit, Whiting, Indiana, USA.
Purpose:	Unstable ground under the bottom slab of a large freight elevator pit solidified to allow concreting and to protect nearby column footing against wash-out and settling.
Chemicals:	Joosten Process chemicals (two-shot).
Methods:	Chemicals injected through pipes into existing bottom slab and pit walls.
Results:	Concrete could safely be poured and solidified sand found safe to transmit heavy column loads through base slab.
Reference:	Chemical Soil Solidification Co., Chicago, Ill., USA.

(22)

Job:	Compressor footing, Air Force Testing Laboratories.
Purpose:	Solidification used to prevent further settling of large, heavily vibrating compressor footing.
Chemicals:	Joosten Process chemicals (two-shot).
Methods:	Base of concrete foundation was widened by injection through pipes reaching deeply under the concrete block.
Results:	Settling was stopped.
Reference:	Chemical Soil Solidification Co., Chicago, Ill., USA.

F. FOUNDATIONS (continued)

(23)

Job: Pier footing, Lawrence, Kansas, USA.

Purpose: Solidification to stop settlement of pier footing carrying heavily vibrating kiln. Material to be grouted was very fine sand above ground water.

Chemicals: Joosten Process chemicals (two-shot).

Materials: Vertical and inclined injection pipes used to reach into core area of subsoil under pier.

Results: No settlement recorded. Two of the corners of the footing were reported raised about 1/2 in.; this is ascribed to the injection into the core under the footing after a peripheral curtain around the footing was solidified.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

(24)

Job: Concrete footing piers, Northern Florida, USA.

Purpose: Solidification of subsoil to increase soil bearing capacity under existing piers.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Pipes driven in at various angles to reach the soil under the footings.

Results: Sufficient area solidified for an ample spreading of the load on the soil to give a uniform load.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

(25)

Job: Kiln piers, Pocatello, Idaho, USA.

Purpose: Stabilization of two 37 ft. kiln piers which had settled unevenly under vibrating loads. Material to be solidified was a powder-like volcanic ash.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Vertical and 45^{o} inclined injection pipes reached an average depth of 42 ft. into core under large concrete base slab of piers.

Results: No further settlement recorded.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

(26)

Job: Wall footings, Milwaukee, Wisconsin, USA.

F. FOUNDATIONS (continued)

Purpose: Protection against settling of wall footings of existing office building, while carrying out large vault excavation about 5 in. away from footings. Material to be grouted was running sand.

Chemicals: Joosten Process chemicals (two-shot).

Methods: 15 ft. injection pipes driven in at an angle under footings.

Results: Satisfactory.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

G. OIL WELLS

(1)

Job: General discussion; chemical methods for shutting off water in oil and gas wells.

Purpose: Intrusion of water into oil wells increases the "lifting" cost of the oil which can be reduced by sealing off the water where well intersects water-bearing strata.

Chemicals: Phenol-formaldehyde plastics; antimony trichloride; silicon tetrachloride.

Methods: Liquid plastic is injected where it polymerizes due to increased temperature. Setting time is controlled by a catalyst. Antimony trichloride injected in concentrated water solution or oil. When solution is diluted upon contact with excess water, insoluble antimony oxychlorides are precipitated. Cost in 1936 was 16 cents a lb. Silicon tetrachloride is injected in oil. Precipitates silica gel when contacted by water. Cost in 1936 was 15 cents a lb. Of these methods, the plastic treatment has been by far the most popular.

Results: Shut-off 99.5 percent effective have been accomplished; however, there is some danger of reducing the oil production.

Reference: "Use of Plastics in Water Control," D. G. Hefley and P. H. Cardwell, The Petroleum Engineer, p. 51, Dec., 1943.

"Plastics in Gas, Water Shut-Offs," J. F. McDonald, World Oil, Vol. 130, p. 108, Mar., 1950.

"Chemical Methods for Shutting Off Water in Oil and Gas Wells," H. T. Kennedy, Transactions, A.I.M.M.E., Petroleum Div., Vol. 118, pp. 177-186, 1936.

(2)

Job: General discussion; chemical methods for consolidating sands in oil and gas wells.

G. OIL WELLS (continued)

Purpose: Loose sand sloughing into oil or gas wells causes increased operating and workover costs which can be reduced by consolidating the incompetent sand formation without reducing excessively the permeability.

Chemicals: Liquid polymers of phenol-formaldehyde.

Methods: Liquid plastic is injected into producing formation where it coats the sand grains and binds them together upon setting. The elevated bottom hole temperature causes the plastic to set, the time being controlled by a catalyst. Permeability is retained as liquid occupies a considerably greater volume than solidified resin it yields.

Results: Estimated that 80 percent of treatments successful; however, oil production is sometimes reduced. Major reasons for failure are: (1) lack of sufficient bottom hole temperature; (2) lack of proper zonal isolation; and (3) lack of proper hole conditioning.

Reference: "Use of Plastics in Consolidating Loose Sand in Wells," R. H. Smith and A. C. Polk, Jr., Petroleum Development and Technology, A.I.M.M.E., pp. 243-249, Mar., 1947.

"Plastics Used to Consolidate Incompetent Formations," P. H. Cardwell, Petroleum Development and Technology, A.I.M.M.E., pp. 174-179, Mar., 1947.

H. RESERVOIRS, WATERWORKS AND WATER WELLS

(1)

Job: Lac Noir Reservoir, France.

Purpose: As early as 1850 a dike had been built at Lac Noir, a small glacial lake, to increase the storage capacity. Leakage occurred when hydroelectric storage involving wide daily variations in level was begun.

Chemicals: Sodium silicate solution; hydrochloric acid; cement; clay.

Methods: Cement-clay-sand used to fill large voids; cement used to seal ordinary cracks; treated clay to grout the coarse sands; silica gel for the fine sands.

Results: Leakage reduced 99.5 percent.

Reference: "Lac Noir Dam, Staunching and Reinforcing," E. Ischy, Third Congress on Large Dams, Stockholm, Question No. 10, R 37, 1948.

(2)

Job: Waterworks at Düsseldorf, Germany.

H. RESERVOIRS, WATERWORKS AND WATER WELLS (continued)

Purpose: Soil was solidified for pipe-line excavations. A layer of solidified soil was left to support the weight of the water pipe.

Chemicals: Not mentioned, but probably Joosten Process.

Methods: Soil was solidified by ordinary method at the bottom of the excavation, forming a solid seal to prevent water from entering reducing uplift. The sheet piling was held at the bottom by the solidified material.

Results: Successful.

Reference: "Die Bedeutung der Grundwasserabsenkung und der Chemischen Bodenverfestigung für die Grundung in Offener Baugrube," Kress and Richardt, Bautechnik, pp. 25-29, Jan. 6, 1933.

"Chemische Bodenverfestigung und Abdichtung," Deutsche Bauzeitung, Vol. 69, pp. 476-478, 1935.

(3)

Job: Water-well in Uioara, Rumania.

Purpose: To solidify area around a 17-ft. diameter fresh-water well and to prevent seepage of salt water into the well. Sealing was accomplished by injecting both cement and chemicals.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Sealing of the concrete well was accomplished by injection through the concrete well-casing into the surrounding sand. Injections were performed from the inside of the well between the depths of 20 to 70 ft. Injections inside were made at 99 points, using 1/2-in. pipe, 2 ft. long. Two Worthington pumps were used, one for each chemical.

Results: The seepage of salt water into the well was reduced 99.7 percent.

Reference: "Chemische Verfestigung und Abdichtung in Rumaenischen Brunnenbau," Ing. O. Bodascher, Bautechnik, pp. 139-140, Mar. 10, 1939.

(4)

Job: Hot water spring in health resort, Teplitz-Schönau, Czechoslovakia.

Purpose: The hot water of the spring was leaking through loose sandstone layers back into its underground flow rather than to the well outlets.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Fissures in the sandstone were sealed by chemical injections through 11 injection pipes driven around the well.

H. RESERVOIRS, WATERWORKS AND WATER WELLS (continued)

Results: The leak of hot-water flow through the well was completely stopped.

Reference: "Chemische Bodenverfestigung und Abdichtung," Karsten, Deutsche Bauzeitung, Vol. 69, pp. 476-478, 1935.

(5)

Job: Sewage settling basin, Waukesha, Wisconsin, USA.

Purpose: Solidification to prevent dangerous uplift while basin is de-watered and to protect inflow and discharge pipes from being bent or broken.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Injection pipes were placed through pre-drilled holes in concrete base slab into subsoil supporting the slab.

Results: Satisfactory bond obtained between existing concrete and piping and the solidified soil. Where pipes were broken a seal was obtained.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

I. TUNNELS AND MINE SHAFTS

(1)

Job: Mine shaft, location unknown.

Purpose: The masonry mine-shaft lining was leaking at the rate of 150 gpm and was to be stopped by solidification.

Chemicals: Sodium silicate solution; sodium chloride solution (two-shot).

Methods: The injection equipment was loaded on the shaft elevator; holes were drilled in the masonry lining; a wooden collar on the injection pipe acted as a packing. Pressures up to 80 atmospheres were used.

Results: Leakage was entirely stopped.

Reference: "Solidifying Gravel, Sand, and Weak Rock," Lars R. Jorgensen, Western Construction News, Nov. 10, 1931.

(2)

Job: Freight tunnel, Chicago, Illinois, USA.

Purpose: Leakage through the cracks, seams, joints, and disintegrated areas of the concrete lining was to be stopped by solidification.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: The chemicals were injected into 14 holes drilled into the worst spots; equipment built around small electric pumps.

I. TUNNELS AND MINE SHAFTS (continued)

Results: Satisfactory.

Reference: "Chemical Joint Sealing and Soil Solidification," C. Martin Riedel, Engineering News-Record, p. 74, Aug. 14, 1941.

(3)

Job: Tunnel for telephone cables in San Francisco, California, USA.

Purpose: Presence of free-running sand made tunneling difficult, even using a splinter method of spiling.

Chemicals: Sodium silicate solution; sodium bicarbonate solution (one-shot).

Methods: Setting time adjusted to about 1 hr. Small duplex piston pump was used. Injection pipe was a 6 to 10-ft. length of 1/2-in. brass pipe.

Results: Satisfactory.

Reference: "Injected Fluid Stops Running Sand," Western Construction News, p. 89, Sept., 1948.

"Solidification of Running Sand," Silicate P's and Q's, Vol. 29, No. 1, p. 2, 1949.

(4)

Job: Railway tunnels, Leyton, England.

Purpose: Soft and waterlogged ground was solidified under building foundation to permit excavation of the tunnels without settlement of the ground.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: Not reported.

Results: Satisfactory.

Reference: "Chemical Consolidation of Ground in Railway Work," H. J. B. Harding and R. Glossop, Railway Gazette, (British), Vol. 72, No. 5, pp. 147-151, Feb. 2, 1940.

(5)

Job: Tube under river, England.

Purpose: Tunnels through running sand were built under compressed air. At river crossings chemical consolidation was used to prevent a "blow."

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: Injection pipes driven from pontoons anchored in the river.

Results: Loss of air through the consolidated ground was negligible.

I. TUNNELS AND MINE SHAFTS (continued)

Reference: "Chemical Consolidation of Ground in Railway Work," H. J. B. Harding and R. Glossop, Railway Gazette, (British), Vol. 72, No. 5, pp. 147-151, Feb. 2, 1940.

(6)

Job: Tube near Mile End Station, England.

Purpose: Pilot tunnels were driven through bad ground, and a hood of consolidated ground was formed for the 12-ft. tunnel driven later.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: Not reported.

Results: Satisfactory—also sealed off water.

Reference: "Chemical Consolidation of Ground in Railway Work," H. J. B. Harding and R. Glossop, Railway Gazette, (British), Vol. 72, No. 5, pp. 147-151, Feb. 2, 1940.

(7)

Job: Coal mine tunnel, Alpha, Illinois, USA.

Purpose: A 4-ft. water-bearing sand stratum under a 50-ft. hydraulic head was sealed off during the driving of the tunnel after dewatering attempts failed.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: Injection pipes driven in pairs, one pipe for each chemical with 8 in. between pipes and 10 to 18 in. between pairs. Hardness of the clay necessitated predrilling of the holes. Heading advanced 8 ft. between injections.

Results: Satisfactory.

Reference: "Chemical Treatment Cures Sick Tunnel," C. Martin Riedel, Construction Methods and Equipment, p. 66, Sept., 1949.

"Rocks out of Sand," Calcium Chloride Association News, Vol. 15, No. 4, pp. 8-9, 1949.

(8)

Job: Alster subway tunnel, Hamburg, Germany.

Purpose: In the construction of the tunnel a retaining wall was required under a busy street intersection. A bad leak developed, causing movement in the wall. The sand and gravel beyond the wall was chemically consolidated and the leak stopped.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: Eleven holes grouted.

I. TUNNELS AND MINE SHAFTS (continued)

Results: Satisfactory.

Reference: "Chemische Abdichtung von Bauwerken und Baugruben," Bautechnik, Vol. 11, pp. 455-457 (Sichardt, W.), 1933.

(9)

Job: Potassium mine at Unterbreizbach (Rhon Mountains), Germany.

Purpose: In running an adit, a passage containing brine at 600 to 1000 psi. pressure was struck. The flow was dammed by filling the adit, and most of the brine flow was carried off through long pipes passing through the dam; 80 cu. ft. of cement grout was then injected through the brine pipes. However, in a few days the dam became wetted through with brine and began to leak. Chemical solidification was then attempted.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: Fifteen holes were bored into the dam and short lengths of pipe cemented in each hole. Eight tons of chemicals were used, with pressures up to 1,150 psi. Brine drain pipes were kept open as long as possible to prevent grouting against the high brine pressures.

Results: Two small drips after 2 1/2 months were sealed by repeated impregnation with the two chemicals under pressures up to 2,100 psi. Since that time the pressure of the brine behind the dam has remained constant at 1,050 psi.

Reference: "The Chemical Solidification of Loose Soils," K. A. Pohl, Engineering Progress, Vol. 13, pp. 85-88, 1932.

"Die Anwendung des Chemischen Verfestigungsverfahrens bei der Abdichtung und bei Schachtdichtungs Arbeiten auf dem Kaliwerk Sachsen-Weimar in Unterbreizback (Röhn)," Kali, Vol. 24, pp. 81-85.

(10)

Job: Sewer diversion tunnel at the Bank-Monument Station, London, England.

Purpose: Well-graded ballast and beds of sand were encountered and caused many difficulties by pouring into tunnel. The material was solidified to facilitate excavation and to eliminate the free-flowing sand.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

I. TUNNELS AND MINE SHAFTS (continued)

Methods: Sodium silicate was injected in measured quantities as the injection pipes were inserted in short drives. When the pipe was withdrawn, a measured quantity of calcium chloride was injected after each short withdrawal. The pipes were driven in the direction of tunneling and radially from the axis of the tunnel. The pressures of pumping varied from 50 to 150 psi.

Results: Excavation time was reduced to 1/3 of that required for untreated sections. Because the solution had 1 1/2 times the specific gravity of H_2O, it displaced any water with which it came in contact, with no subsequent dilution. This work was successful.

Reference: "New Methods of Excavating Quicksand," Sir Henry Japp, Engineer, Vol. 156, No. 4059-4060, pp. 422-425, Nov. 3, 1933. (Very complete description.)

"Chemische Bodenverfestigung und Abdichtung in Tunnelbau," Verein-Deutscher Ingenieure, Zeitschrift, Vol. 77, pp. 905, 907, 1933.

(11)

Job: Mine shaft at Ollerton, Nottinghamshire, England.

Purpose: Triasic and Permian sandstones and Permian limestone were encountered.

Chemicals: Sodium silicate solution; sulphate of alumina (injected either separately or combined).

Methods: Holes 60 to 120 ft. in depth were drilled in floor for high-pressure grouting pipes. Annual space between hole and pipe was sealed off. Then by using a high-pressure pump (up to 3,000 psi.) chemicals were injected to seal off small fissures. Cement grout then followed, to seal off larger voids. Excavation was thereafter carried out under normal conditions.

Results: Water was sealed off in zones containing large quantities of water; the chemicals acted as a lubricant for the grout by forming a gel. This permitted much less pumping pressure and sealed even the smallest pores.

Reference: "Francois Cementation Method," R. D. Hall, Coal Age, Vol. 33, pp. 106-107, Feb., 1928.

(12)

Job: Tunnel for storm water from the Lake Merced district to the Pacific Ocean, San Francisco, California, USA.

Purpose: Tunneling being through a loose dry beach sand, it was extremely difficult to excavate. By stabilizing, drilling operations were made easier and pressure on the breastboards and supporting timbers caused by the flowing sand was reduced.

I. TUNNELS AND MINE SHAFTS (continued)

Chemicals: Sodium silicate solution; sodium bicarbonate (one-shot).

Methods: 10-ft. needles were driven ahead in a radial pattern about the centerline of the tunnel. The chemical solution was controlled so as to give the soil supporting strength and still maintain ease of excavation.

Results: Amount of combined solution required varied from 30 to 45 gals. per cu. yd. of solidified sand. Using pressure of 200 psi. at needle and injection time of 1 min., ground was solidified for a radius of 1 ft.

Reference: "Chemical Stabilization of Sand Speeds Driving of Ten-Foot Tunnel," Roy E. Wright, Engineering News-Record, Vol. 143, p. 42, Aug. 4, 1949.

(13)

Job: Mine, Shannon, New Birmingham, Alabama, USA.

Purpose: It was found that the hard rock encountered contained many water-bearing fissures. These fissures contained so much water that drilling operations had to be discontinued in some parts of the mine.

Chemicals: Not reported.

Methods: Instead of ordinary low-pressure pumps, continuous high pressure (2000 to 3000 psi.) injection pumps were used. The chemicals for producing a fine colloidal filling were first injected into the finer fissures ahead of the grout, forced under high pressure.

Results: Successful. Chemical grouting stopped water flow in fissures.

Reference: "Francois Grouting Process," Engineering News-Record, Vol. 102, p. 853, May, 1929.

(14)

Job: Tunnel for subway, Moscow, Russia.

Purpose: Materials such as quicksand (extremely wet) and drenaceous clays were found to make excavation difficult. The chemicals were used to solidify these materials for excavation and to render them impervious to water. This method was also used successfully to seal off water flowing through leached-out concrete retaining walls.

Chemicals: Sodium silicate solution; calcium chloride solution; sometimes cement was added (two-shot).

I. TUNNELS AND MINE SHAFTS (continued)

Methods: Process started with an injection of water glass followed by a brine of calcium chloride of common salt; sometimes cement was added. Grout was injected with ordinary pump through 1-in. pipes, 15 to 30 ft. long: perforated with 0.08 in. holes at injection end. The dosage was controlled by means of meters.

Results: Satisfactory; quicksand was solidified into a shell 3 ft. thick; within 15-20 min. the treated soil attained a compressive strength of 280-850 psi. The injected concrete was impermeable to water.

Reference: "The Moscow Subway," Engineering News-Record, Vol. 116, pp. 515-521, Apr. 9, 1936.

"Chemical Consolidation in the Moscow Subway," L. Jorgensen, Engineering News-Record, Vol. 117, p. 483, Oct. 1, 1936.

(15)

Job: Mine, near Birmingham, Alabama, USA.

Purpose: The limestone and rock, containing many water-bearing faults and fissures, had to be sealed off before mining procedures could be carried on. The chemical grouting was used in conjunction with cement grout to accomplish the desired sealing. This mine contained a record amount of water; and at some grouting locations, water flowed at 600 gal./min. with a pressure head up to 2200 ft.

Chemicals: Sodium silicate solution; sulphate of alumina (two-shot).

Methods: Boreholes were drilled into rock, placement pipes were inserted and cemented into place. Then an injection by two separate pipes—one pumping sodium silicate and the other sulphate of alumina—was made to form a colloidal gel; grout was next pumped under continuous pressure (2500 psi.) to seal off larger fault crevices and fissures; 851.5 bbl. of limestone dust and cement were pumped into larger fissures before signs of filling were seen.

Results: Successful.

Reference: "Tunneling a Water-Bearing Fault by Cementation," B. J. Hall, Engineering News-Record, Vol. 102, p. 874, May 30, 1929. (A very interesting article.)

(16)

Job: Tunnel for subway, London, England.

Purpose: Solidification of sand above the tunnel profile was (1) to prevent movement of sand (2) to reduce seepage of water and (3) reduce cost of tunnel shoring.

I. TUNNELS AND MINE SHAFTS (continued)

Chemicals: Joosten Process chemicals (two-shot).

Methods: A 2-ft. sand layer above the crown of the 9-ft. diameter tunnel was solidified by injecting first sodium silicate while the injection pipe was driven into the layer, followed by injection of calcium chloride during the withdrawal of the pipe. Injection pipes were 15 ft. long; 100-ft. length of tunnel was solidified.

Results: The solidification was successful. Cement grouting, which was also tried, was not satisfactory since it would not penetrate into the sand.

Reference: "Chemische Bodenverfestigung und Abdichtung in Tunnelbau," Dr. Ing. W. Sichardt, Verein-Deutscher Ingenieure Zeitschrift, pp. 905-907, Aug. 19, 1933.

"Chemische Bodenverfestigung und Abdichtung," Deutsche Bauzeitung, Vol. 69, pp. 476-478, 1935.

(17)

Job: Daggafontein and Brakpan Mines, Europe.

Purpose: As the No. 2 shaft was sunk a quantity of water was encountered which hindered the operations considerably. Cement grouting was used to seal off this flow of water.

Chemicals: Francois and Portier Grouting Process (cement-grout solution).

Methods: As each fissure or hole bearing water was struck, cement grout was applied to seal off the flow. Progress was observed by the rise in pressure of the pumping operations. When a pressure of 2000 psi. was reached, pumping was stopped, and the cement was allowed to set.

Results: The holes being 180 ft. deep, 55 tons of cement were used. The operation proved satisfactory.

Reference: "The Francois and Portier Grouting Process," A. H. Krynauw, Engineering and Contracting, Vol. 51, p. 461, Apr. 30, 1919.

(18)

Job: Dearborn Street Subway, Chicago, Illinois, USA.

Purpose: Solidify immersed sand under high hydraulic head (above 50. ft.) and stop infiltration of sand and water into a sewer with leaky joints situated beneath the concrete base slab of the subway tube.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Two rows of injection pipes driven in at an angle in order to reach the bottom soil under the sewer. The sewer joints were completely encircled with a water-tight wrap of concrete.

I. TUNNELS AND MINE SHAFTS (continued)

Results: No leakage reported.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

(19)

Job: Dearborn Street Subway service shaft, Chicago, Illinois, USA.

Purpose: Large, concrete-lined service shaft with leaking seams, cracks and honey-comb areas to be solidified to prevent an inflow of approximately 400 gals./min.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Injection holes intersecting cracks were drilled to receive grout packers. Chemicals I and II pumped in by hand pumps; maximum pressure of 600 psi. was achieved by injecting very small quantities of solution.

Results: Leakage stopped.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

(20)

Job: Service shafts, Detroit, Michigan, USA.

Purpose: To stop heavy leaks in deep service shafts in salt mine by chemical injection.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Chemicals injected by pressure into leaking concrete lining and into voids behind the lining. Work had to be carried out from hoist cages due to lack of working space.

Results: Leakage stopped.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

(21)

Job: Ammunition igloos, Eastern USA.

Purpose: 80-ft. long, dome-like roofs were covered with a fill consisting of sandy soil, loam and broken stone. Solidification used to prevent leakage through fine cracks and fissures in fill where a cobaltic coating was apparently damaged or dried out.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Horizontal pipes were driven into the fill and chemicals were injected as the pipes were withdrawn.

Results: Leakages stopped.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

I. TUNNELS AND MINE SHAFTS (continued)

(22)

Job: Storm sewer work shaft, Evergreen Park, Illinois, USA.

Purpose: Solidification work to prevent in-flow of ground water into shaft and to protect mining operations against cave-in.

Chemicals: Joosten Process chemicals (two-shot).

Methods: 30-ft. long injection pipes were driven concentrically around the existing shaft.

Results: Complete seal was achieved.

Reference: Chemical Soil Solidification Co., Chicago, Ill., USA.

(23)

Job: Pennsylvania Turnpike tunnels, Pennsylvania, USA.

Purpose: To fill large voids behind the concrete tunnel lining, and, at the same time, to prevent ground water leakage through construction joints and cracks in that lining.

Chemicals: Not stated (two-shot).

Methods: The tunnel lining was first backfilled with Lelite sand. The layer adjacent to the tunnel lining was then solidified by the injection of two chemicals mixed separately outside the tunnel, transported to the injection site, and combined at the nozzle. Cracks and construction joints were further sealed by drilling holes only part way through the lining at 12 to 18-inch intervals. Injectors placed in these holes forced the solutions into the cracks or joints, making them impervious to ground water.

Results: 90 percent of the leakage was stopped.

Reference: "Chemical Sealing Stops Leakage in Tunnels of Pennsylvania Turnpike," C. W. Stickler, Jr., and A. Allen, Jr., Civil Engineer, Vol. 24, No. 11, pp. 723-724, Nov., 1954.

J. VIADUCTS

(1)

Job: Viaduct in Bremen, Germany.

Purpose: Viaduct was being damaged by settlement.

Chemicals: Sodium silicate solution; calcium chloride solution (two-shot).

Methods: The soil was consolidated into blocks under the viaduct columns.

Results: Apparently satisfactory.

J. VIADUCTS (continued)

Reference: "The Development of the Joosten Process of Soil Consolidation During a Ten-Year Period of Practical Application," Dr. Eng. Adolph Mast, Bautechnik, Vol. 16, No. 21, Waterways Experiment Station Translation No. 39-15, May 20, 1938.

K. GENERAL DISCUSSIONS AND FIELD EXPERIMENTS

(1)

Job: Experimental excavation, below water table, Germany.

Purpose: In this experimental excavation, the soil outside the steel sheet-piling was solidified to increase the strength and reduce the permeability of the soil. The experimental excavation was 11 1/2 x 29 1/2 ft.

Chemicals: Joosten Process chemicals (two-shot).

Methods: Sheet-piling was driven to a depth of 28 ft. The top 1 1/2 ft. of soil (above the water table) between the sheet piling was removed. The soil along the outside of the sheet-piling was solidified to about 3 ft. below the bottom of the sheet-piling, after which the soil excavation was completed.

Results: This experiment was made by the Siemens-Bauunion Co. The results were satisfactory; only small amount of water entered the excavation, and no noticeable uplift of the bottom occurred.

Reference: "Erfahrungen mit der Chemischen Bodenverfestigung und Anwendungsmöglichkeiten des Verfahrens," W. Sichardt, Bautechnik, pp. 181-186, Mar. 18, 1950.

(2)

Job: Underpinning, Queen Anne's Alcove, London, England, Leaning Tower of Pisa, Italy, and other jobs.

Purpose: This article deals with chemical grouting; it is a general discussion of the process and results. In all cases, the purpose was to stabilize load-carrying soils or to seal off fissures and cracks bearing unwanted water flows.

Chemicals: Sodium silicate solution; calcium chloride solution, or a gas (two-shot).

Methods: Low-pressure pumps generally used to inject chemicals by two-shot method. Chemicals were injected usually in stages of 22 in. in depth, with a radius of solidification of about 10 - 16 in.

Results: Good. Author does not believe this should replace concrete construction where concrete can be used. Strengths ranged from 250 to 1100 psi. Chemical resistance is comparable to that of concrete.

K. GENERAL DISCUSSIONS AND FIELD EXPERIMENTS (continued)

Reference: "Soil Solidification," C. M. Riedel, Midwest Engineer, Vol. 1, No. 6, p. 6, Feb., 1949.

(3)

Job: Solidification experiment on underground railroad pit on the Neukölln Railroad, Germany.

Purpose: The railroad pit was excavated in a sandy material. This job was used as an experiment for chemical solidification. The sand behind the walls and below the bottom of the pit was solidified, and the extent and completeness of solidification was then determined by excavation. The pit was 10 ft. deep.

Chemicals: Joosten Process chemicals (two-shot).

Methods: 1/2-in. pipes were driven horizontally through the wooden shoring into the sandy material. Pipes were pointed and perforated. Pipes were also driven vertically below bottom of pit which was below the water table. Injection pressures ranged from 15 to 100 psi.

Results: Chemicals had a tendency to leak back through the shoring, and it was necessary to seal the shoring. The solidification was successful. Solidified sand below the water table appeared as strong as that above.

Reference: "Versteinung Loser Sande als Grundungsverfahren," E. Biermann, Deutsche Bauzeitung, Vol. 61, Supplement: Konstruktion und Ausfuhrung, pp. 177-183, 1927.

(4)

Job: Discussion of field experiences with chemical grouting.

Purpose: To improve stability and impermeability of sandy soils, sands, and gravels.

Chemicals: Sodium silicate (two-shot and one-shot); calcium acrylate; acrylamide and methylene-bis-acrylamide.

Methods: Chemical injected into soil acts as a binder and fills voids. Various field procedures are described.

Results: Except for one application, chemical grouting was successful.

Reference: "Field Experiences with Chemical Grouting," M. Polivka, L. P. Witte, and J. P. Gnaedinger. Paper presented at the Annual Meeting of the A.S.C.E. in New York City, New York, Oct., 1954.

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J. VIADUCTS

K. GENERAL DISCUSSIONS AND FIELD EXPERIMENTS

IV. PATENT ABSTRACTS

(Chronological 1890 - 1954)

1890 M. Garvey -- METHOD OF STOPPING SEAMS
No. 442,037; 12/2/1890

Deposits cartridge of paraffine; pressure is applied to cartridge, forcing paraffine into seams.

1906 N. Mehner -- PROCESS OF SOLIDIFYING EARTHY GROUND
No. 829,664; 8/28/1906

Injection of a mineral substance in liquid condition, melted gypsum alone or with other material (chloride of magnesium) only example given.

1919 John C. Swan -- APPARATUS TO HEAT ROCK AND INTRODUCE SEALING VAPOR
No. 1,307,207; 6/17/1919

The patent covers the use of apparatus to heat rock and introduce a sealing vapor into the rock.

1921 John C. Swan -- METHOD OF EXCLUDING EXTRANEOUS FLUIDS FROM WELLS
No. 1,379,657; 5/31/21

This patent covers an injection procedure whereby a suitable heating medium, such as hot water, is used to heat the pores underground to such a degree that a fluid sealing compound forced into the material will solidify. For the sealing compound Swan uses the vapor of naphthalene, either by itself or mixed with a carrier gas such as natural gas and with or without superheated stream.

1922 Ronald Van Auken Mills -- PROCESS OF EXCLUDING WATER FROM OIL AND GAS WELLS
No. 1,421,706; 7/4/22

This patent covers the process of introducing into wells, porous sands, or other porous rocks or rock-forming materials, one or more soluble chemical reagents, either as solids, liquids, gases or muds, dry or in aqueous or other solutions, free or in containers; and under necessary pressure that is practical, so that the said reagent or reagents come in contact with and react chemically with each other, react with the rock wall materials of the well, or with the dissolved constituents of natural waters or other solutions in the wells and interstices of porous rock in such manner as to cause chemical and physical precipitation in the wells and rock interstices.

Mills lists seven examples of his reaction, as follows: (1) sodium silicate with calcium chloride, (2) sodium silicate with magnesium

1922 (contd) chloride, (3) sodium silicate with hydrochloric acid, (4) sodium carbonate or sodium bicarbonate with calcium chloride, (5) sodium sulphate with barium chloride, (6) calcium sulphate with sodium silicate, (7) calcium oxide with sodium silicate.

Albert Francois -- METHOD OF MAKING FISSURED WATER-BEARING STRATA WATERTIGHT
No. 1,430,306; 9/26/22

Francois has applied for or obtained patents on this process in the following countries: Belgium (No. 265,867), Great Britain (No. 8,482), Germany, Poland, Austria, Czechoslovakia, Hungary, France (No. 470,528).

This patent covers a process of filling fissures, consisting of initially injecting colloidal material and thereafter of injecting cement milk. The colloidal material, consisting of a colloidal precipitate formed from commercial silicate of soda and sulphate of alumina, is used as a treatment to prepare the fissures to receive the cement. Francois claims that the colloidal material acts somewhat like a lubricant, which nullifies the frictional resistance of the walls of the fissures to the penetration of the cement, in the case of small fissures; in the case of large fissures, the colloidal substance appears to penetrate between the particles and prevent the cement from setting in a compact mass, whereas the slight consistency of the colloidal material offers no obstacle to the subsequent penetration of cement.

1927 Harold E. Potts -- PROCESS OF CEMENTATION IN THE GROUND
No. 1,635,500; 7/12/27

This patent covers the use of a caustic alkali solution as a pre-injection to a cement suspension. Potts claims that this prepares the borehole so that the particles of cement will pass freely thereto. He finds that caustic soda is the best of the caustic alkalies.

1930 George W. Christians -- METHOD AND APPARATUS FOR SEALING CREVICES IN ROCK FORMATIONS OR THE LIKE
No. 1,763,219; 6/10/30

In Christians' patents No. 1,327,268 and 1,327,269, dated January 6, 1920, he disclosed a method for sealing crevices in rock formations and the like by means of a thermoplastic material. This method was based on using a siphon to get the material from the heating kettle into the hole.

The present patent covers a feeding method and apparatus employing a straight gravity feed from the upper portion of the heating kettle to the pump, and permits the gas to escape from the feeding pipe. While this patent discusses thermoplastic materials, the only one that the author actually names is hot asphalt.

1931 Michael Muller -- PROCESS OF CHEMICALLY SOLIDIFYING EARTH
No. 1,815,876; 7/21/31

Muller's process consists of first saturating the earth with silicic acid-containing substances and then applying chlorine gas. The result is silicic acid, which combines with the quartz-containing constituents of the earth.

C. W. Christians -- APPARATUS FOR SEALING CREVICES IN ROCK FORMATIONS OR THE LIKE
No. 1,820,347; 8/25/31

This patent, like the previous one (1,763,219) is on apparatus used to inject a thermoplastic material. The patent covers a perforated pipe which is placed in a passageway which communicates with the crevice, crack, or fissure to be sealed. It also covers the use of a heating wire connected to the lower end of the conduit; this heating wire is used to keep the thermoplastic material plastic until it is in the formation to be sealed.

Carl Zemlin -- PROCESS OF SOLIDIFYING LAYERS OF GROUND AND SIMILAR MASSES
No. 1,820,722; 8/25/31

This patent covers the use of a single uniform chemical solution which reacts with the soil to bring about the solidification. The only example which Zemlin gives and the only claim which he has covers the use of injecting hydrofluoric acid to react with the silica in the soil. This reaction gives silica fluoride, which in turn continues to act on the earth salts and acids to set silica free again and tends to cement together the solid particles of soil.

Hugo Joosten -- PROCESS OF SOLIDIFYING PERMEABLE ROCK, LOOSELY SPREAD MASSES OR BUILDING STRUCTURES
No. 1,827,238; 10/13/31

This patent covers the injection of silicic acid-containing materials, followed by the injection of a gas which reacts with said materials to form silicic acid, which gels in situ from the nascent state and thus integrates the treated mass. The only gas suggested is carbon dioxide. Joosten also claims the injection of gel-forming chemicals followed by a gas.

1932 G. W. Christians -- METHOD AND APPARATUS FOR SEALING CREVICES IN ROCK FORMATIONS OR THE LIKE
No. 1,858,952; 5/17/32

This patent covers improvements in the technique and apparatus used to inject thermoplastic material. In previous patents only asphalt has been mentioned, but this patent mentions pitch and sulphur as well as asphalt.

The main new feature covered by this patent is to enclose the perforated portion of the injection pipe with a puncturable material before placing the pipe in the hole. The thermoplastic substance is forced by pressure to puncture the covering, and then the material flows into the crevices.

1933 W. G. Brown -- CAULKING COMPOUND AND METHOD OF APPLICATION
No. 1,931,643; 10/24/33

Cement plus pulverized oxidizable metal which swells on oxidation.

1935 Wilhelm Klie -- PROCESS OF AND APPARATUS FOR FILLING CRACKS AND CREVICES
No. 1,987,626; 1/15/35

This patent covers a procedure and apparatus for the pressure injection of subsurface material. The procedure consists of employing two pipes in such a way that the outer pipe seals off the borehole and the inside pipe, or pressure pipe, can then force the injection juice under pressure into the material to be grouted.

Wilhelm Klie -- METHOD AND MEANS FOR FILLING IN CONCRETE, ROCKS, AND THE LIKE
No. 1,987,958; 1/15/35

This patent is an improvement over No. 1,987,626 in that only one pipe is required to perform the injection.

John J. Grebe and Sylvia M. Stoesser -- TREATMENT OF DEEP WELLS
No. 1,998,756; 4/23/35
(Assigned to the Dow Chemical Co.)

This patent covers the injection of solution or dispersion of organic material which forms gel in situ. Suggests protein material + hot water and organic material followed by second fluid such as acid.

H. T. Kennedy and H. C. Lawton -- METHOD OF PLUGGING STRATA IN WELLS
No. 2,019,908; 11/5/35

This patent covers a process of impermeabilization in which is injected a solution of a silicon halogen compound or sometimes of a titanium halogen compound in a suitable non-aqueous liquid such as oil, the compound forming a plugging, cementitious deposit on hydrolysis in the interstices of the strata. The solution is kept in the strata in the presence of water to bring about such hydrolysis and to seal up the strata. Two compounds which have been found to work are silicon tetrachloride and titanium tetrachloride. The first of the two is regarded as the best. In general, any organic solvent can be used, provided it is inert, that is, not of a type which reacts with the silicon tetrachloride.

J. R. Jorgensen -- METHOD OF GROUTING BY CHEMICAL MEANS
No. 2,025,948; 12/31/35

This patent covers the process of solidifying loose sand, gravel, and other granular materials surrounding piles, pile clusters, tower legs, fills or walls. It comprises injection into a system of

1935 (contd) conduits embedded in or attached to the piles of a solution of sodium silicate followed by calcium chloride in combination with a gel-forming gas. One of Jorgensen's claims covers the use of calcium chloride in combination with carbon dioxide, the combination being pre-cooled. The main item in this patent seems to be the use of the conduits, which are put inside or alongside of piles, etc.

Loomis, Teplitz and Ambrose -- METHOD OF TREATING WELLS
No. 2,034,347; 3/17/36

Injection of a cellulose material which will deposit when diluted with water.

1936 Jan van Hulst -- PROCESS OF SOLIDIFYING SOILS
No. 2,051,505; 8/18/36

This patent covers a process of injecting successive aqueous dispersions of a bituminous substance, each dispersion being less stable. In one of his examples, he first introduces approximately 20 liters of a stable dispersion which is prepared by dispersing 1 part by weight of bitumen in 2 parts by weight of a 5/10 percent soap solution, after which 1 percent of casein, calculated on the weight of the bitumen, is added for stabilization. After the injection of this dispersion, 200 liters of a bitumen dispersion prepared in the same manner without the addition of the casein, is introduced through each pipe. By decreasing successively the stability of the injected dispersion the author breaks up the dispersion in situ.

1937 J. McKay -- METHOD OF WATERPROOFING STRUCTURES
No. 2,071,758; 2/23/37

Emulsion of organic plastic medium and a de-emulsifying agent is injected into drilled holes.

Jan van Hulst -- PROCESS FOR SOLIDIFYING EARTH
No. 2,075,244; 3/30/37

This patent has three features—in any application any one or any comgination of these features may be used.

The first feature is to place in the ground a quantity of coarse material such as gravel, gravel stone, stone chippings, or rock aggregate around the place where the injection juice is to be introduced. This is supposed to aid penetration.

The second feature covers a process consisting of introducing an aqueous dispersion of a bituminous substance such as asphalt and causing this dispersion to coagulate at a desired place by suitably controlling the stability of the dispersion. The stability is controlled by adding to the dispersion coagulation-promoting agents such as electrolytes.

The third feature of this patent considers using a mixture of an aqueous bitumen dispersion with a finely divided colloidal substance such as various types of clays (bentonite, refractory,

1937 (contd) potter's, fullers earth), water glass, silicic acid gel, diatomaceous earth, Cassel earth and other substances containing humic acids, gelatine, glue, etc.

C. E. Cannon -- PLUGGING WATER SANDS BY A SOAP PRECIPITATE
No. 2,079,431; 5/4/37

Injection of a soap solution, allowing the solution to mix with oil and salt water, thereby salting out the soap to plug the soil pores.

Hugo Joosten -- PROCESS FOR SOLIDIFYING SOILS
No. 2,081,541; 5/25/37

Joosten uses the injection of a single concentrated solution containing the silicic acid sol in an unstable or labile state. For this purpose the composition specifically described is that which is formed from a concentrated solution of an alkali silicate by first adding a suitable precipitating metal salt solution, particularly such as that of soluble zinc salts (for example zinc chloride or sulphate), and then bringing the precipitate thus obtained again to solution by adding ammonia or substances containing ammonia, or by previously admixing such ammonia and thereby preventing the formation of the precipitate. The Joosten Process consists of first injecting this unstable gel simultaneously with the introduction of the material which reacts with the ammonia or expells it, or followed by the introduction of a material which releases the ammonia. He suggests a number of chemicals for expelling the ammonia, such as hydrochloric acid, acid salts such as sodium bicarbonate or bisulphate, copper salts, iron salts, etc. The main gas he suggests is carbonic acid gas. A mixture of air and carbon dioxide is carbonic acid gas. Joosten also covers the subsequent introduction of a highly concentrated solution of calcium chloride.

John J. Grebe -- METHOD OF PREVENTING INFILTRATION IN WELLS
No. 2,090,626; 8/24/37

Injection of a soluble silicate and a water-soluble soap.

1938 Carroll Irons -- METHOD OF PLUGGING POROUS STRATA IN WELLS
No. 2,121,036; 6/21/38

For brine water: Injection of rubber latex which is coagulated by salt water.

James G. Vail -- CONSOLIDATION OF POROUS MATERIALS
No. 2,131,338; 9/27/38

This patent covers a process consisting of impregnation with an unstable silicious colloidal liquid having an alkaline reaction in the state in incipient gel formation, and the said liquid is allowed to gel in situ. Control of the time of setting can be accomplished by dilution or control of pH, for example. The best mixture reported consists of a solution of sodium silicate containing not substantially less than two mols of silica to one mol of sodium oxide with a solution of sodium aluminate, the concentration of said solutions being adjusted to produce, upon admixture, an

1938 (contd) unstable dilute liquor setting to a full volume alkaline gel within a period of the order of thirty minutes.

1939 H. T. Kennedy -- PROCESS OF SHUTTING OFF WATER OR OTHER EXTRANEOUS FLUID IN OIL WELLS
No. 2,146,480; 2/7/39

Injection of a material which is hydrolyzed upon contact with water to form an insoluble solid matter. Examples: metal salt, salt of antimony, arsenic, bismuth, tin and iron, antimony trichloride.

John J. Grebe -- No. 2,152,307; 3/28/39
(Assigned to The Dow Chemical Co.)

An alkali phosphate and a water-soluble soap, the latter in excess, are introduced to plug the pores of water strata in a well. The treating solution may be forced into the pores by a hydrostatic head of oil.

John J. Grebe -- No. 2,152,308; 3/28/39
(Assigned to The Dow Chemical Co.)

A water-soluble aluminate and a water-soluble soap, the latter in excess, are introduced into wells to plug the pores of water strata.

T. H. Dunn -- No. 2,156,219; 4/25/39
(Assigned to Stanolind Oil and Gas Co.)

A soluble lead salt in solution is injected to plug the pores of brine-bearing strata. The lead chloride precipitated is said to be an especially effective plugging agent.

T. H. Dunn -- No. 2,156,220; 4/25/39
(Assigned to Stanolind Oil and Gas Co.)

A solution of magnesium salt, and after it a solution of an alkaline hydroxide, are forced into water-bearing strata of a well, and excess pressure is held on the system sufficiently long for the chemicals to react and plug the pores with a voluminous precipitate of magnesium hydroxide.

J. van Hulst and G. H. van Leeuwen -- PROCESS OF SOLIDIFYING SOILS
No. 2,158,025; 5/9/39

This patent covers a process of impermeabilization which comprises a mixture of an aqueous dispersion of a bituminous substance with finely divided filling substances that will not exert any substantial flocculating or coagulating action on said dispersion. The patent mentions a number of colloidal substances which may be added, for example, bentonite, refractory blue clay, potter's clay and the like, water soluble hydroxides of polyvalent metals such as aluminum, iron, or tin hydroxide, colloidal or peptized organic substances such as gelatin, vegetable glue, humus or humic acid-containing substances (examples: Cassel earth, polysaccharides such as gum arabic, agar-agar, starch and the like).

1938 (contd) The inventors claim that the presence of the colloidal matter results in larger conglomerates being built up, and therefore the effectiveness of the injection liquid is increased. In other words, an 8 percent solution with added colloids might be as effective as a 30 percent bitumen dispersion without them.

F. A. Bent, A. G. Loomis, and H. C. Lawton -- No. 2,169,458; 8/15/39 (Assigned to Shell Dev. Co.)

Metal alcoholates are introduced into wells to form water-insoluble hydroxide precipitates for sealing off gas and water formations. Slowly hydrolyzing alcoholates are preferred, e.g., aluminum secondary emyl alcoholate and the aluminum alcoholate of ethylene glycol.

R. J. Ball -- No. 2,174,027; 9/26/39

This method claims to seal off wells by introducing a soluble alginate into the well and then introducing a compound ($CaCl_2$ preferred but Al $(SO_4)_3$ or $FeCl_3$ may be used) which reacts with the soluble alginate to form an insoluble alginate in situ. It may also be used with cement in which case a mixture is made with 90 percent cement and 10 percent sodium alginate solution (8 lb/gal water) which is pumped into the hole. A calcium chloride solution is then pumped into the hole. Long stringy chemical fibers are formed in situ which aid in shutting off oil, gas, or water wells as desired.

T. G. Malmberg -- PROCESS FOR SOLIDIFYING PERMEABLE MASSES No. 2,176,266; 10/17/39

This patent covers a process for tightening and consolidating rock formations, sand and gravel layers, porous concrete, dam structures, shafts and drill holes, etc., comprising the steps of preparing aqueous mixtures of an alkali silicate and a water soluble acid salt of a weak acid, injecting said mixture into the material to be treated prior to any appreciable coagulation reaction having taken place in the mixture, and letting the reaction between the alkali silicate and the acid salt proceed with the formation of silica gel as an injection material. For acid salts the inventor names sodium bicarbonate, sodium tetraborate, and sodium bisulphite.

NOTE: Claims 1, 2, and 3 of Patent No. 2,176,266 are disclaimed.

1940 D. C. Matthews -- No. 2,186,875; 1/9/1940

A porous well formation is sealed with a compound that will automatically liquefy at some time after reaching maximum thickness, e.g., a fermentable organic gel plug. Cement and casing are then put over the surface of the compound, which may be, for example, at the upper end of a producing formation, and the cement is allowed to set before the gel has time to liquefy.

1940 Hans A. Reimers -- COMPOSITION FOR STOPPING LEAKS
(contd) No. 2,188,311; 1/30/40

This patent covers the composition of aqueous solutions which are injected and then subjected to hydrolysis. The primary intent is to stop leaks in pipes, especially metal, but the patent states that structures of concrete, earthenware, earth and rock can be treated also. For the soluble silicate Reimers uses an alkali metal silicate such as potassium or sodium silicate, preferably sodium silicate, in a concentration of about 20 percent of silicate by weight in a stock solution. For the metal salt he used a sulphate, chloride, or nitrate of aluminum, cadmium, chromium, divalent copper, trivalent iron, divalent manganese, trivalent tin, and zinc. He gives percentage mixtures for the various soluble silicates and metallic salts. After the silicates and salt are in the structure to be made water-tight, a potential of about 1 to 4 volts is passed through the liquid, with the result that a water-resistant, semi-rigid gel is deposited at the anode.

Jan van Hulst -- PROCESS OF FIXING AND IMPERMEABILIZING SOIL MASSES
No. 2,190,003; 2/13/40

This patent covers the injection of dispersion of bitumen and/or rubber latex or any other coagulable binding agent which coagulates in situ.

A. C. Hamilton, Jr. -- No. 2,191,652; 2/27/40
(Assigned to U. S. Gypsum Co.)

Claim 2 - The method of sealing off an opening in the earth which comprises the steps of first filling it with water, then forcing into it a slug or column of potentially hardenable calcined gypsum gauged with water and characterized by a definitely predetermined time of set, permitting water to recede in the opening so as to cause the slug to travel downwardly in the opening until it reaches the point to be sealed, the time of setting of the calcined gypsum being so predetermined by admixture with sufficient set-controlling substances as to cause it to set at about the time when the slug reaches the said point.

G. H. van Leeuwen -- PROCESS OF IMPERMEABILIZING, TIGHTENING, OR CONSOLIDATING GROUNDS AND OTHER EARTHY AND STONY MASSES AND STRUCTURES
No. 2,197,843; 4/23/40

This process consists of injecting a substance which is capable of swelling through a solvating agent, the particles of which substance are coated with a substance repelling the solvating agent, the swelling of said particles being effected in the mass under treatment by attracting or adsorbing or combining with or wetting by the said solvating agent.

Where the solvating agent consists of water or an aqueous solution of dispersion, the swelling substance may comprise such

1940 (contd) things as colloidal clays, hydroxides of polyvalent metals, silicic acid, aluminates or other salts capable of swelling with water or of forming liquid crystals, and such organic colloids as polysaccharides such as cellulose or starch, gum arabic, agar-agar, lipoides, proteins such as casein and albumen, organic dyestuffs and the like. Wherever the solvate consists of organic liquids such as oil, hydrocarbons, chlorinated hydrocarbons, alcohols, carbon disulfide, and the like, the swelling substance may comprise, for example, rubber, balata, shellac, drying oil polymerization products, factis, nitrocellulose, acetyl cellulose, soaps and the like, which are termed oleophile colloids.

The substances repelling the solvating agent, such as water, which are used in combination with the hydrophile colloids, are particularly oils, such as mineral oils, oil fractions and residues, tar oils and the like. Such repellent substances are called hydrophobic. In the case of the solvating agents consisting of organic liquids, such as oils, which are used in conjunction with the oleophile colloids, the substance repelling the solvating agent may be an oleophobic substance, in most cases water or an aqueous liquid.

Van Leeuwen gives a number of examples of injection fluids.

W. B. Lerch, C. H. Mathis, and E. J. Gatchell -- No. 2,198,120; 4/23/40 (Assigned to Phillips Petroleum Co.)

The space between the well bore and casing is plugged with a gel-forming mixture of 50 parts sodium silicate, 50 parts water, 100 parts hydrochloric acid, and carbon black.

F. A. Bent, A. G. Loomis, and H. C. Lawton -- No. 2,200,710; 5/14/40 (Assigned to Shell Dev. Co.)

Somewhat selective plugging of salt-water-bearing formations is done by injecting a water-soluble fluosilicate to form insoluble sodium fluosilicate in the presence of brine. If desired, further precipitates can be formed by injection of a soluble hydroxide such as NaOH or KOH, with the resultant precipitation of silicic acid and calcium fluoride. Since these reactions take place rather slowly and the carrying agent (water) is immiscible with oil, the action is selective and can be made to occur deep in the formations.

H. C. Lawton and A. G. Loomis -- No. 2,204,233; 6/11/40 (Assigned to Shell Dev. Co.)

The invention is concerned with an improved method for solidifying unstable formations and selectively scaling off water and gas formations transversed by wells. The steps of the process consist of introducing into the well an inert non-aqueous solvent miscible with water and oil and forcing it into the formation. The solvent is allowed to dissolve the formation waters, and the solvent together with the waters dissolved therein are then withdrawn

1940 (contd) from the formation and the well. The water removed with the solvent is replaced with water in the water-bearing layers and with oil in the oil-bearing layers. A sealing agent forming oil water insoluble compounds by hydrolysis is introduced into the formation and a sealing precipitate is formed within the water-bearing layer. The solvents used to carry out this process contain a small amount of a surface-tension reducing agents such as amine bases. This invention specified pyridin in its last claim.

H. A. Reimers -- No. 2,207,759; 7/16/40
(Assigned to The Dow Chemical Co.)

Well strata are plugged with liquid silicate compositions which set to gels in a known time, after a prior injection of acid has destroyed interfering substances.

H. C. Lawton -- No. 2,208,766; 7/23/40
(Assigned to Shell Dev. Co.)

To form a gel plug for porous formations, a solution of a soluble silicate and some weak base or salt of a weak base is introduced to form a gel within the formation. Data show controllable times for gel formation with pyridin, ammonium sulfate, ammonium acetate and others.

A. C. Hamilton, Jr. - No. 2,210,545; 8/6/40
(Assigned to U. S. Gypsum Co.)

In sealing formations in a well with cement, a substance accelerating the set of the mixture is added in increasing amounts as the mixture is pumped in. The accelerating agents include seed crystals of calcium sulfate for gypsum, and sodium silicate for portland cement. (Equivalent to Canadian Patent No. 408,671 of Nov. 17, 1942)

H. T. Byck, J. W. Freeland, H. C. Lawton -- No. 2,211,688; 8/13/40
(Assigned to Shell Dev. Co.)

A method is claimed for solidifying unstable formations and sealing off water and gas layers traversed by wells which comprises circulating in the borehole, a drilling fluid comprising a water-soluble alginate, an alkali hydroxide and a water-soluble substance selected from the group: (a) organic materials of feebly acidic properties and (b) alkali metal pyro-and metaphosphates. Porous or unstable formations are sealed by solidification or by forming an insoluble sheath or plug on the walls of the borehole or within the adjacent formations.

W. B. Lerch, T. M. White, and E. J. Gatchell -- No. 2,214,423; 9/10/40
(Assigned to Phillips Petroleum Co.)

Porous well formations are plugged with a mixture of 5 percent highly unsaturated fish oil, 45 percent petroleum distillate, 45 percent carbon tetrachloride and 5 percent sulfur monochloride, which sets forming a resinous plug. Claims are broad to a

1940 (contd) combination of highly unsaturated animal oil, saturated hydrocarbon, heavy organic solvent, and sulfur monochloride.

C. R. Irons -- No. 2,219,319; 10/29/40
(Assigned to The Dow Chemical Co.)

A liquid treating agent, and particularly an agent such as liquid styrene for sealing off water-bearing formations, is placed in a well. A non-penetrating liquid of lower specific gravity, which may be a solution of an organic jelly, is placed above the treating liquid, and pressure is applied to the second liquid to force the treating agent into the pores of the formation.

Orie N. Maness -- No. 2,219,325; 10/29/40
(Assigned to The Dow Chemical Co.)

A method of cementing a selected portion along a perforated well liner is shown. The cement or other sealing material, e.g., vinylidene chloride and styrene polymers, is guided into position by the use of a liquid mixture incapable of penetrating porous formations and having substantially the same specific gravity as the sealing material. After the cement is positioned, a non-penetrating liquid carrying a filler is forced into the well to seal perforations in the liner and prevent backflow of cement. Suitable non-penetrating liquids are dispersions of bentonite or starch in water. Fillers are mica flakes, hemp fibers, film scraps and the like.

H. Limburg -- No. 2,223,789; 12/3/40
(assigned to Shell Dev. Co.)

Water strata in a well are sealed by introducing a solution of asphalt in an organic, water-miscible solvent, adapted to precipitate the asphalt when diluted with water. As solvents, pyridine and cresol are suggested. An earlier treatment of strata with alkali sulfonate or napthanate is advantageous.

S. P. Hart -- No. 2,224,120; 12/3/40
(Assigned to Texas Co.)

A method is claimed to seal off underground crevices and fissures which communicate with the well bore by plugging them with a plastic mass comprising a casein putty of the following composition: 5 percent casein plastic plug hydrated lime.

1941 Charles Langer -- PROCESS OF STANCHING AND CONSOLIDATING POROUS MASSES
No. 2,227,653; 1/7/41

This patent covers the injection of a single solution consisting of water glass and a reactive agent comprising an acid and a strong coagulant. The existing pH of the sodium silicate is decreased by the addition of an acid in order to obtain a weaker alkaline solution. By further adding a suitable salt of a heavy metal (iron, copper, lead, zinc and the like) as an electrolyte, the latter solution is destroyed and coagulates to a gel. By decreasing the pH value the sodium silicate solution becomes more sensitive and

1941 (contd) the coagulation to a gel in the ground or other mass being treated may be produced at any time desired by means of a correspondingly accurate quantity of electrolyte. The particular chemicals which appear to be the best, since the author specified these, are sodium silicate, hydrochloric acid and copper sulphate.

H. T. Kennedy and A. J. Teplitz -- No. 2,229,177; 1/21/41
(Assigned to Gulf Res. and Dev. Co.)

In a method of selectively plugging water producing formations in a well a first solution is injected into all formations of a character which will prevent plug formation by a solution introduced subsequently. The well is then allowed to produce. The plugging inhibitor will be entirely flushed from the water formations, but a part of it will be retained in the connate water in the oil formation. Finally a plugging reagent is introduced and allowed to set, but due to the presence of the inhibitor in the connate water of the oil sand, little plugging occurs there. The plugging agent which has remained fluid is removed and the well is ready for production. As an example, the first solution may be phenyglycine in acetone and the second methyl silicate with HCl to control the setting time. (Equivalent to their Canada No. 405,292, Patent 6-9-1942)

Charles S. Ackley -- METHOD OF SOLIDIFYING POROUS EARTH MATERIALS
No. 2,232,898; 2/25/41

Injection of molten sulphur.

Charles S. Ackley -- METHOD OF RENDERING EARTH MATERIALS SOLID
No. 2,235,695; 3/18/41

Injection of molten sulphur.

W. B. Lerch, C. H. Mathis, and E. J. Gatchell -- No. 2,236,147; 3/25/41
(Assigned to Phillips Petroleum Co.)

Formations in wells are plugged by introducing a liquid gel-forming material comprising a mixture of one part sodium silicate diluted with one part of a water solution containing 3-1/2 parts hydrochloric acid and 19 parts of sodium bisulfate solution. The acid and bisulfate delay the premature setting of the gel until the solution has penetrated the formation where it reacts with salts and acids to form a gel which later solidifies.

L. C. Chamberlain and H. A. Robinson -- No. 2,238,930; 4/22/41
(Assigned to The Dow Chemical Co.)

The invention relates to methods of reducing the permeability of earth or rock formations with the formation of a plugging deposit within certain strata penetrated by the bore, thus preventing infiltration of water by introducing into the formation a water-miscible solution of a stabilizing agent (salts of organic acids) and

1941 (contd) then a non-aqueous water-miscible solution of a metal salt capable of forming a precipitate of a basic compound by reaction with an aqueous alkaline material. The stabilizing agent whereby the precipitation of the basic compounds is delayed in the water-bearing stratum and substantially prevented in the other gas-bearing stratum.

C. H. Mathis -- No. 2,252,271; 8/12/41
(Assigned to Phillips Petroleum Co.)

A method of sealing cracks or porous formations by injection of a resin-forming liquid is claimed, which is particularly suitable for plugging limestone and dolomitic materials due to its non-acid character. This particular resin is formed from an ester of a dicarboxylic acid and a polyhydric alcohol, condensed or copolymerized with or without a vinyl derivative, using benzoyl peroxide as a catalyst. The amount of catalyst added controls the time of setting of the fluid to a solid resin after it is placed in the porous formation. Being a non-acid, carbon dioxide which might otherwise be evolved in a reaction with the limestone, cannot impair the effectiveness of plug formation.

L. S. Wertz -- No. 2,254,252; 9/2/41

A process is claimed for solidifying and filling the small interstices of a porous medium. To a thin slurry of portland cement and water are added a finely divided material containing acidic colloidal silica and a suspension stabilizing agent. The finely divided material may be fly ash or some types of hydraulically active blast furnace slag, or natural siliceous materials generally classified as pozzolana. Many types of stabilizers may be used, such as mineral oil with salts of fatty acids, alginates, bentonites, and the like. Such a composition is claimed to be highly fluid and capable of being forced into small capillaries without losing water and solidifying prematurely.

F. A. Bent and A. G. Loomis -- No. 2,259,875; 10/21/41
(Assigned to Shell Dev. Co.)

The sealing properties of a non-aqueous drilling fluid are improved by adding a solution of an ester of silicon. Upon contact with formation waters the organic silicon compounds hydrolyze and deposit silica, and other precipitates which act as plugging agents. The advantage over the use of inorganic silicon solutions is that all the products of the hydrolysis or condensation are non-corrosive. Alcohol is among the solvents proposed for the ester of silicon.

F. A. Bent and A. G. Loomis -- No. 2,265,962; 12/9/41
(Assigned to Shell Dev. Co.)

A process for selectively plugging water formations in an oil well is claimed. The plugging agent is an ester of silicon which hydrolyzes upon contact with water in the formation to deposit silica and complex silicon compounds. The rate of hydrolysis is controllable by changing the pH of the treating solution, and/or by

1941 (contd) selection of the particular ester, or its concentration. One of many possible compounds of this class is ethyl-ortho-silicate.

Norris Johnston -- No. 2,267,683; 12/23/41
(Assigned to Socony Vacuum Oil Co.)

A fusible metal is used in drilling operations for the recovery of small objects such as bit rollers, or where it is desired to make a temporary water shut-off. Ordinarily the melting point is adjusted in compounding the alloy so that the metal will be solid at the bottom hole temperature, but can be melted by a circulating fluid of higher temperature. It is placed in the hole in solid form and melted by a hot circulating fluid, whereupon small metal objects are enclosed, or it seals off water formations. When allowed to cool, it solidifies and can be drilled through or fished out of the hole. An alloy containing bismuth, lead, tin, cadmium, and antimony is mentioned as an example.

L. C. Chamberlain -- No. 2,267,855; 12/30/41
(Assigned to The Dow Chemical Co.)

A conditioning liquid is claimed for injection ahead of the selective plugging agent in making a partial salt water shutoff. A non-acid surface tension reducer such as a water-soluble alcohol, ketone, phenol, cresol, xylenol, or the like may be used. The effect of this is to permit the water sand to be penetrated by the plugging agent more freely, with the result that more precipitate can be formed therein. Also, less plugging fluid will penetrate the oil sand and require washing out.

1942 H. T. Kennedy -- No. 2,270,006; 1/13/42
(Assigned to Gulf Research and Dev. Co.)

In a method of sealing porous water-bearing strata by injecting a compound which forms a plug upon contact with water, the plugging agent used is one which takes considerable time to set, and the initiation of setting is variably controlled by addition of an accelerator. The sealing agents suggested are compounds of polyvalent metals carrying at least one OR group, where R stands for an alkyl or aryl radical. Examples are zinc ethylate Zn $(OC_2H_4)_2$, aluminum triphenolate $Al(OC_6H_4)_3$, and tri-chlor stannic ethylate $SnCl_3OC_2H_4$. Accelerators may be silicon tetra chloride, or metal chlorides which form acid upon going into solution, such as $FeCl_3$, $ZnCl_2$, $CuCl_2$.

C. Irons and S. M. Stoesser -- No. 2,274,297; 2/24/42
(Assigned to The Dow Chemical Co.)

Plugging of porous formations, sealing around formation packers, shutting off bottom water, and similar operations are carried out using a resin-forming liquid which is put in place or forced into the formations and allowed to set to a solid resin. Styrene may be used with or without a catalyst, and pure or diluted with oil, depending on well temperatures, setting time desired, etc. Polymerization of the resin to a solid body may be carried out at well temperatures or by artificial heating. Vinylidene chloride and phenolformaldehyde solutions are also claimed.

1942 J. B. Stone and A. J. Teplitz -- EARTH CONSOLIDATION
(contd) No. 2,281,810; 5/5/42

This patent covers a method wherein pervious earth formations are injected with an acid organic-silicate sol in a state of incipient gellation and adapted to set to a gel after an interval of time. The gel time of the sol is controlled by the adjustment of the acidity by incorporation in the sol of a polybasic acid. Enough polybasic acid is used to delay the setting of the soil in the presence of calcium carbonate to between 1/4 of an hour and 2 hours. The sol claimed is one comprised of methyl silicate mixed with water. The polybasic acids mentioned are acids of phosphorus, oxalic acid, and citric acid.

John J. Grebe -- TREATMENT OF WELLS
No. 2,294,294; 8/25/42
(Assigned to The Dow Chemical Co.)

This patent covers the injection of a material which by polymerization, addition or condensation, forms in situ a synthetic resin.

C. R. Irons -- No. 2,298,129; 10/6/42
(Assigned to The Dow Chemical Co.)

A fusible metal alloy is used for plugging of porous well formations which are producing undesirable fluids such as water or brine. Pieces of a low melting point alloy are dropped into the well to a point opposite the formation to be plugged, an electric heating element is lowered to melt them, pressure is applied to displace the molten metal into the formation, after which the heater is withdrawn and the metal allowed to harden and become set in place.

G. H. van Leeuwen -- No. 2,300,325; 10/27/42

Plugging of a permeable body such as a body of earth or a stratum in a well is accomplished by impregnating the earth or stratum with oleophile colloids suspended in a liquid which can cause them to swell, but from which they are temporarily protected by an oleophobic substance that does not cause swelling. Some oleophile colloids mentioned are rubber, balata, shellac, nitrocellulose, acetyl cellulose, and the like. The substance causing swelling may be oils, chlorinated hydrocarbons, alcohols, carbon disulfide, etc.; and the oleophobic protective substance is preferably plain water. Not claimed are the converse: water-swellable colloids protected by films of oil and the like.

1943 C. H. Mathis and Carl Rampacek -- No. 2,307,843; 1/12/43
(Assigned to Phillips Petroleum Co.)

Plugging of formations in wells is performed using a resin-forming liquid prepared by mixing water, thiourea, and furfural, allowing the mixture to undergo partial condensation in the presence of hydrochloric acid added as a catalyst, then adding an alkali sufficient to reduce the pH to between 5.5 and 6.5, and

1943 (contd) finally placing the mixture in the formation where further condensation to a solid resin will occur. Setting time may be controlled by the amount of HCl used. Resins prepared in this way are particularly suited for use in limestone when otherwise a reaction with excess acid would occur, producing gaseous products which would impair the strength and sealing qualities of the set resin.

Louis S. Wertz -- COMPOSITION FOR DENSIFYING POROUS MASSES AND STRUCTURES
No. 2,313,107; 3/9/43

This patent covers injection of a slurry consisting of portland cement and a filler having sufficient acidic colloidal silica to retard the gelling of the cement, a relatively small amount of a lubricating and a plasticizing agent, and a relatively small amount of protective colloid. Wertz uses acidic colloidal silica in the amount of from 1/2 to about twice the amount of cement by weight, and up to 2 percent of at least 1 oleaginous material. The only plasticizer or protective colloid mentioned is ammonium stearate in the amount of up to 2 percent of the weight of the cement.

Louis S. Wertz -- PROCESS FOR FILLING CAVITIES
No. 2,313,110; 3/9/43

This patent employs a process similar to that patented under Wertz's patent No. 2,313,107, namely a readily flowable mixture of portland cement, a filling material having acidic colloidal silica, and a lubricating agent. The present patent covers the use of this mixture and aggregate to repair disintegrated areas on the surfaces of structures. The author claims that this mixture with the aggregate will give a very strong bond between itself and the old structure.

G. H. van Leeuwen -- No. 2,319,020; 5/11/43
(Assigned to Shell Dev. Co.)

A process is claimed for sealing porous masses of earth or strata in wells. Two substances which react chemically with each other to form a precipitate are injected, one of the substances in colloidal form being coated with a protective film which retards the precipitate - forming reaction. An example of this is a solution of 10 percent colophony in an aromatic gasoline and, on the other hand, a 5 percent aqueous aluminum chloride solution. The acids contained in the colophony are converted into aluminum salts which swell by absorbing gasoline.

D. L. Mitchell, Harry Marks, and H. C. Beene -- No. 2,320,633; 6/1/43

A water-sealing composition for oil wells comprises by volume 70 percent oil well cement, 15 percent unslaked lime, 10 percent beidellite, and 5 percent iron oxide. To this is added water sufficient to make a thin mortar. When desired, calcium chloride up to about 1 percent may be added as an accelerator, paper pulp up to 3 percent as an additional sealing agent may also be added,

1943 (contd) and for tight formations tannic acid from 1/2-1 percent may be used as a thinning agent. These compositions are said to be incapable of setting in an oil formation and are, therefore, selective in shutting off water.

F. D. Sullivan -- No. 2,320,954; 6/1/43

A method of solidifying and plugging soil or strata in well is claimed, comprising forcing water and tetra sodium pyrophosphate into the soil or strata, followed by a clay which reacts with the tetra sodium pyrophosphate in place to cause swelling of the clay. A non-swelling montmorillonite may be transformed into a swelling montmorillonite similar to benonite by treatment with .25 percent to 10 percent of this pyrophosphate.

John J. Grebe and L. C. Chamberlain -- No. 2,321,138; 6/8/43
(Assigned to The Dow Chemical Co.)

An improvement in the plugging of wells by introducing an electrolyte which forms an insoluble precipitate with a naturally occurring or subsequently introduced ion comprises electrolyzing the solutions in the formation by passing a direct electric current through them. It is said that the effect of the electrolysis is to accelerate the rate of diffusion of the ions so that they sooner become mixed and precipitate throughout a larger portion of the formation.

C. H. Mathis and Carl Rampacek -- No. 2,321,761; 6/15/43
(Assigned to Phillips Petroleum Co.)

A synthetic resin suitable for use in wells and particularly in limestone strata (where strong acids cannot be used) comprises a mixture of furfural, a urethane, and a hydrochloric acid catalyst to control the time of setting. As the mixture has a pH of about 7, limestone formations will not be attacked by it.

Abraham B. Miller -- No. 2,323,928; 7/13/43
(Assigned to Hercules Powder Co.)

Substantially petroleum-hydrocarbon insoluble pine wood resin is used as a soil stabilization agent, alone or in conjunction with other stabilizers such as $CaCl_2$. The amount used may be between 0.12 and 10 percent and preferably is between 0.25 and 2.5 percent.

Abraham B. Miller -- No. 2,323,929; 7/13/43
(Assigned to Hercules Powder Co.)

A method of stabilizing soils by incorporating 0.2 to 10 percent of a substantially hydrocarbon insoluble pine wood resin as an aqueous suspension formed by mixing the resin with dilute alkali and saponifying a minor proportion of the resin.

1943 G. H. van Leeuwen -- PROCESS OF IMPERMEABILIZING, TIGHTENING, OR CONSOLIDATING GROUNDS AND OTHER EARTHY AND STONY MASSES AND STRUCTURES
(contd)
No. 2,329,148; 9/7/43

This process consists of injecting into the voids a substance which is capable of swelling through a solvating agent, the particles of which substance are coated with a substance repelling the solvating agent, the swelling of said particles being effected in the mass under treatment by attracting, or adsorbing, or combining with, or wetting by the said solvating agent. In other words, a suspension of a colloid which will swell is put into the soil in a liquid carrier. If the colloid swells by being solvated with water, it is protected so that the solvation rate is slow. The rate of solvation and swelling can be varied by considerable control of the acidity and the polarity of the substances used. Van Leeuwen gives eight examples:

1. A colloidal clay is coated with a film of an extract obtained in the treatment of kerosene with liquid sulfur dioxide.
2. An aqueous dispersion of casein which has been treated with lubricating oil fractions rich in aromatics.
3. A rubber latex, kerosene, and benzene.
4. Rubber latex and an aqueous dispersion of iron napthenate, aluminum palmitate.

Numbers 5 and 7 involve a rubber latex. Number 6 is a resin solution in gasoline and an aqueous aluminum chloride solution. Number 8 is finely powdered aluminum sulphate suspended in 2 parts of spindle oil and 5 parts of a 20 percent water glass solution.

Lewis A. Schmidt, Jr. -- SUBSTRATUM WATER CONTROL
No. 2,329,223; 9/14/43

This patent is applicable to those cases where the size or the extent of the cavities is too great to permit successful grouting with normal material. It consists in effect of constructing a vertical continuous concrete wall by means of interconnected grout holes.

H. A. Reimers -- No. 2,330,145; 9/21/43
(Assigned to The Dow Chemical Co.)

A sealing composition for well formations is claimed comprising 8 to 16 percent by weight of sodium silicate and 4.7 to 20.5 percent sulfuric acid. By varying the ratios of these two components in a water solution a great deal of control is possible in the time required for setting to a firm gel. An extensive table is given showing, for different temperatures and compositions of the mixture, the minutes duration of a pumpable state and the final set strength in grams. By reference to this table it should be possible to choose the composition best suited to a given well condition.

1943 R. E. Davis -- CONCRETE CONSTRUCTION PROCESS
(contd) No. 2,331,311; 10/12/43

This patent describes a method of constructing a monolithic concrete structure free from stresses or cracks due to thermal changes resulting from hydration of the cement.

Milton Williams -- No. 2,332,822; 10/26/43
(Assigned to Standard Oil Dev. Co.)

Plugging agents for shutting off water strata in oil wells, which are readily removable by acidizing, are disclosed and claimed. The preferred agents are mixtures of arsenates or phosphates with salts of aluminum, calcium, cobalt, chromium, copper, iron, magnesium, manganese, or zinc. These precipitate as gels, which are readily soluble. A chart is given of setting time vs. temperatures for various mixtures of chromium acetate and disodium arsenate, and for mixtures of chromium acetate and disodium phosphate. The feature of acid removability should reduce the hazards usually associated with the use of gel forming materials in that if oil production is accidentally shut off it can be restored.

1944 K. L. Vonder Ahe and H. C. Zweifel -- No. 2,338,217; 1/4/44
(Assigned to Richfield Oil Corp.)

Loss of circulation in drilling wells is combatted by spotting a quantity of ammoniacal solution of casein opposite the porous section of the well, subsequently introducing formaldehyde into the casein, and forcing the mixture into the formations. This sets up to a hard resinous plug which seals the well against further drilling fluid loss. Preferred proportions are 3 to 4 parts by weight ammonium hydroxide, 100 parts casein, and 200 to 600 parts water. Formaldehyde to harden this runs from 20 to 60 parts per 100 parts casein. Penetration is controlled in some degree by varying the viscosity of the ammoniacal casein solution.

S. E. Buckley and G. G. Wrightsman -- No. 2,338,799; 1/11/44
(Assigned to Standard Oil Dev. Co.)

A selective plugging process for wells is claimed in which an oil-soluble, water-insoluble resin is employed and is injected into the formations while in an oily intermediate stage of condensation. After initial setting, the well is produced for several days to wash out any of the resin which does not set because of oil contamination. The well is then shut down again and a second charge of the resin injected to increase the effectiveness of the plugging. In an example water production was reduced from 82 percent to 63 percent by the second resin injection. Preferred resins are formed by condensing alkylated phenols with formaldehyde in the presence of a catalyst, or they may be modified alkyd, vinyl, ures-aldehyde or styrene resins.

1944 W. B. Lerch, C. H. Mathis, and E. J. Gatchell -- No. 2,345,611; 4/4/44
(contd) (Assigned to Phillips Petroleum Co.)

Claims are asserted to the use of aldehyde-urea synthetic resins for plugging off water formations in wells. A preferred composition comprises thiourea and furfural with concentrated HCl as a catalyst in sufficient quantity to delay the time of set of the mixture until it is in place in the formation to be plugged.

Joseph C. Becker -- No. 2,348,320; 5/9/44
(Assigned to Atlantic Refining Co.)

A composition suitable for lining of salt water pits (reservoirs for accumulation of salt water separated from crude oil) is claimed comprising an intimate mixture of bitumen, top soil and sawdust. A wide range of percentages of the individual components is cited.

H. C. Lawton -- No. 2,348,484; 5/9/44
(Assigned to Shell Dev. Co.)

Acaroid resin (yacca gum, or grass tree gum) in an alcohol solution is injected into a formation to be plugged and the well is then allowed to produce slowly. Water or brine leaches the alcohol out depositing the resin which, under the influence of the formation heat, sets up to a solid plug. 65 percent to 35 percent methyl alcohol is a typical preferred treating solution. An advantage claimed for this resin over most plugging agents is that it sets in the presence of residual oil left in the formations where the water shut-off is to be made.

W. B. Lerch, C. H. Mathis, and E. J. Gatchell -- No. 2,349,181; 5/16/44
(Assigned to Phillips Petroleum Co.)

A liquid resin-forming mixture of furfural and thiourea is claimed as a substitute for cement slurry in cementing casing. The setting time is controlled by varying the amount of hydrochloric acid used as a setting catalyst, and a filler may be added to provide bulk without greatly adding to the material cost. Bentonite, wood fiber, fine sand, carbon black and other similar nonreactive materials are disclosed as fillers. A relatively inexpensive resin disclosed but not claimed comprises furfural, caustic oil (a waste product from caustic washing of cracked distillate) catalyst, and filler.

Abraham B. Miller (Hazel E. Miller, administratrix) -- No. 2,357,124; 8/29/44
(Assigned to Hercules Powder Co.)

A method of soil stabilization comprising admixing therewith 1 to 10 percent of a mixture of tall oil and a pine wood resin substantially insoluble in petroleum hydrocarbons in the form of an aqueous suspension formed by saponifying a minor proportion of the mixture with dilute alkali.

1944 John W. Poulter -- No. 2,363,018; 11/21/44
(contd) (Assigned to Koehring Co.)

Soil in a pavement subgrade is stabilized by displacing any water therefrom and impregnating it with a mixture of loam, binder (e.g., portland cement), bituminous material and water.

K. H. Andersen -- No. 2,365,039; 12/12/44
(Assigned to Case, Pomeroy and Co.)

In a flooding or gas-repressuring system to produce oil from oil-bearing sands a method is proposed for selectively plugging permeable oil-depleted strata or thief sands in an oil well in order that the driving fluid will flow through less permeable oil-producing strata. By this method after a well is emptied of water, a first dilute solution of sodium silicate and a bicarbonate salt, e.g., sodium bicarbonate, which solution gels in 10-20 hours, is slowly flowed into the well under such regulated pressure that it selectively enters the pores of the more permeable strata. A second solution, which contains a higher concentration of bicarbonate salt such that gelling occurs in 6-8 hours, is then flowed into the well, whereby a viscous deposit is formed in the treated strata to withstand the driving-fluid pressures to which the "dam" will be subjected.

M. C. Leverett and G. G. Wrightsman -- No. 2,366,036; 12/26/44
(Assigned to Standard Oil Dev. Co.)

Water-oil ratios from producing wells are reduced by pumping an agent, for example, an alkylated phenol-formaldehyde resin, into the water and oil sands in a liquid state, this resin being capable of setting at bottom-hole conditions to a solid which is soluble in oil but insoluble in water. Other resins include glyptol types rendered oil soluble by inclusion of vegetable oils, glycerol, any phthalic anhydride condensed with oleic acid or linseed oil, certain vinyl resins, modified urea aldehyde resins, and certain styrene resins.

1945 A. B. Miller -- No. 2,369,682; 2/20/45
(Assigned to Hercules Powder Co.)

Soil is stabilized by dispersing therein 0.05 to 2.0 percent sodium acid abietate, a loose combination of one mole of sodium abietate with three moles of abietic acid.

A. B. Miller -- No. 2,370,983; 3/6/45
(Assigned to Hercules Powder Co.)

A method of soil stabilization which comprises mixing with the soil 1 to 5 percent portland cement and 0.5 to 2.0 percent of pine wood resin. The resin, as described in the specification, may be wood resin, gum resin, various heated, oxidized or hydrogenated resins, or the resin obtained by extracting stump pine with benzene or toluene, distilling off the solvent, and extracting the residue with gasoline to dissolve the resin and leave the

1945 (contd) "petroleum hydrocarbon-insoluble resin" which appears to be the preferred material for this use.

A. B. Miller -- SOIL STABILIZATION
No. 2,377,639; 6/5/45

Soil plus alkali soap of a petroleum hydrocarbon insoluble pine wood resin.

L. C. Chamberlain -- No. 2,378,687; 6/19/45
(Assigned to The Dow Chemical Co.)

Fragmented magnesium is pressed into a crevice, fissure, or well bore, and treated with a corrosive solution such as sodium chloride brine, which forms water-insoluble corrosion products such as magnesium oxy-chloride with the magnesium. These corrosion products of magnesium generally occupy more volume than the original metal and are hard, strong, impervious substances. Therefore, they press against the sides of the crevice, fissure, or well bore, and form hard, strong impervious plugs.

G. G. Wrightsman and S. E. Buckley -- No. 2,378,817; 6/19/45
(Assigned to Standard Oil Dev. Co.)

Incompetent petroleum-producing formations are consolidated by a process consisting of the following steps: (1) Drying the formations, (2) Adding a material to make the formation wettable by a cementing material, (3) Forcing the cementing material such as a plastic into the formation to coat the grains and cement them together, (4) Either forcing a neutral liquid such as oil or water into the formation, or producing the well to open passages through the cementing material, and (5) Allowing the cementing material to set to bond the grains together.

A. D. Garrison -- No. 2,379,516; 7/3/45
(Assigned to Texaco Dev. Corp.)

The quantity of water produced by an oil well is reduced by forcing oil-wettable, water-repellant material into the formations at around 3000 psi surface pressure. The formations may be fractured, sheets of the material near the oil-water interface preventing vertical migration of water, and sheets of material in the oil zone aiding in conducting oil to the well.

1946 Delmar H. Larsen -- PROCESS OF COMPACTING OR SEALING FORMATIONS
No. 2,393,173; 1/15/46

Injection of a suspension of a deflocculated, non-colloidal, non-gelling, non-swelling material, e.g., commercially ground barytes of 325 mesh.

H. T. Kennedy and P. L. Bassett -- No. 2,411,793; 11/26/46
(Assigned to Res. and Dev. Co.)

Water-oil ratios of producing wells are improved by squeezing with a plastic or silica gel or sol of sufficient gel strength to

1946 (contd) permit penetration of only large pores and fractures where water usually occurs.

Louis S. Wertz -- No. 2,434,302; 1/13/48

The present invention relates to a process of reinforcing and solidifying porous masses, such as porous concrete, rock, masonry, pieces of packed aggregate and the like, which includes forcing an improved flowable or intrusion composition into the void spaces of such porous masses to increase their strength and water-tightness.

1948 R. M. Hodgson -- PROCESS FOR THE HARDENING OF SOIL AND THE LIKE
No. 2,437,387; 3/9/48

Solution of a material from calcium and magnesium chlorides, then a solution of a material from sodium and potassium hydroxides, finally solution of sodium silicate.

Cary R. Wagner -- No. 2,439,833; 4/20/48
(Assigned to Phillips Petroleum Co.)

A formation may be plugged off to water flow by injecting an aqueous solution of sodium carboxymetyl cellulose and a sufficient amount of a salt to produce a water insoluble precipitate. The precipitate may be removed by treating with one of the strong bases.

S. S. Kurtz, Jr., and J. S. Sweely -- No. 2,457,160; 12/28/48
(Assigned to Sun Oil Co.)

According to the method disclosed, prior disadvantages encountered in sealing off certain undesirable well formations with cement and various types of resins are overcome by introducing into the zone to be sealed off, an aqueous suspension of a partially condensed thermo-setting resin. Resins of the phenol-formaldehyde class are typical of those recommended. In application of the aqueous resin suspension, the latter is introduced at the point of the formation to be sealed off and forms a resin plug at the face of the bed which upon application of heat condenses to a hard layer non-porous to drilling fluids. The suspension employed is capable of forming a sheath at the face of the porous body without appreciable penetration of the resin into the body even though the latter has pores or voids substantially larger than the dispersed resin particles.

1949 P. H. Cardwell -- No. 2,485,527; 10/18/49
(Assigned to The Dow Chemical Co.)

Permeable formations penetrated by a well bore are plugged by injecting a mixture of two partial condensation products. One is the partial reaction product of an aldehyde with an alkylated phenol. The other is the partial reaction product of an aldehyde, a phenol, and a polyphydroxy benzene selected from the group consisting of phloroglucinol and resorcinol. The mixture reacts rapidly at normal well temperatures with little shrinkage to form a slid plug in the permeable formation.

1949 M. C. Dailey -- No. 2,492,212; 12/27/49
(contd) (Assigned to U. S. Gypsum Co.)

A water-resistant resinous material, typically a melamine-formaldehyde resin, is incorporated in a calcined gypsum cement slurry for cementing or sealing wells and well formations. The resin prevents dissolution of the set cement by water. Aldehyde condensation products of triozine are also taught and claimed. Twenty-five claims on composition and methods of use.

J. C. Seaver, A. F. Shepard, and F. W. Less -- No. 2,527,581
(Assigned to Durez Plastics and Chem., Inc.)

Water zones in an oil well are plugged by injecting into the water zone a plastic which will set up rapidly. Many phenol-formaldehyde resins are disclosed, but a mixture of phenol-aldehyde plastic, resorcinol-aldehyde plastic and free aldehyde is claimed.

1951 G. G. Wrightsman -- No. 2,556,863; 6/12/51
(Assigned to Standard Oil Dev. Co.)

Diphenyl-thiourea dissolved in a hydrolyzable solvent such as the alkyl esters is injected in the formation where the increased temperature causes hydrolysis of the solvent, thereby precipitating the solute to plug the formation.

Chester N. White -- SEALING AGENTS
No. 2,570,892; 10/9/51
(Assigned to Sun Oil Co.)

This patent covers sealing agents consisting of a mixture of a polysaccharide and an aqueous suspension of dispersed condensation-type thermosetting resin, partially condensed. Heat is applied to set.

August Holmes -- No. 2,575,170; 11/13/51
(Assigned to Standard Oil Dev. Co.)

This invention relates to improvements in soil stabilization and particularly in treating and consolidating mixtures of coarse and fine aggregates to form rigid, semi-rigid and even plastic layers to serve as base supports for bituminous pavements. According to this invention, higher strength and higher dry soil densities in soil can be obtained by impregnating the soil with non-aqueous liquids. Liquids particularly suited for this purpose are oils of low viscosity, not over 100 ssu at 100°F. and other liquids showing relatively low solubility in water.

1952 G. G. Wrightsman -- METHOD FOR CONSOLIDATING OR FOR PLUGGING SANDS
No. 2,595,184;4/29/52

Form fluid of formaldehyde, unsubstituted hydroxy aromatic compound and a strongly alkaline catalyst. (2 to 10 percent by weight of fluid for catalyst content)

1952 W. C. Blackburn -- PROCESS FOR PREPARING A GROUTING FLUID
(contd) No. 2,618,570; 11/18/52

Fifty volumes of tetraethyl or the silicate, 30 volumes of 95 per-cent ethyl alcohol, one volume of water. Let stand 24 hours (to hydrolyze some of the silicate) then mix with aqueous alkaline solution.

1953 de Mello, Hauser and Lambe -- No. 2,651,619; 9/8/53

Acrylate of polyvalent metal and catalyst system.

1954 Paul L. Menaul -- METHOD OF SEALING POROUS FORMATIONS
No. 2,670,048; 2/23/54

Patent covers injection dispersion of acrylic resin in hydrocarbon followed by injection anionic fluid to coagulate or precipitate resin.

FOREIGN PATENTS

1939 F. A. Bent, A. G. Loomis, and H. C. Lawton -- No. 382,960; 7/25/39; Canada
(Assigned to Shell Dev. Co.)

A metallic alcoholate is introduced into a well to react and form a water insoluble hydroxide for sealing off porous formation. The alcoholate preferably contains at least some derivative of slowly hydrolyzing, higher alcoholates. Alternate charges of a non-aqueous alcoholate solution, oil, and water may be introduced so as to cause gradual mingling reaction.

John J. Grebe and S. M. Stoesser -- No. 385,751; 12/19/39; Canada
(Assigned to The Dow Chemical Co.)

Porous well formations are plugged with a water-insoluble viscid material and a water-soluble organic solvent, e.g., hardwood pitch and acetone.

John J. Grebe -- No. 385,752; 12/19/39; Canada
(Assigned to The Dow Chemical Co.)

Porous well formations are plugged with portland cement or the like in suspension in a gas stream.

1940 Carroll Irons and Sylvia M. Stoesser -- No. 386,475; 1/23/40; Canada
(Assigned to The Dow Chemical Co.)

A resin-forming liquid is introduced in an earth or rock formation and transformed to solid resin by heat or other agency.

1927 Hugo Joosten -- METHOD OF STABILIZATION OF MOUNTAIN LAYERS
No. 441,622; 3/9/27; Germany

The patent covers a method of stabilization of quartz-containing earth based on the reaction of silicic acid-containing material and soluble salts or acids, with or without filling materials. The reaction produces silicic acid in situ, which improves the stability of the mass.

1939 N. V. de Bataafsche Pet. Mij. -- No. 849,712; 11/30/39; France

Water-bearing formations in a well are plugged up by treatment with a fluosilicate and an alkali, e.g., with a fluosilicate of Ca, Mg, Pb, Fe, aniline, diphenylamine, and others, and an alkali, such as NH_4OH, NaOH, KOH. A number of products precipitate, including some by interaction with natural brine components. As equivalents of fluosilicates, the fluotitanates and a few analogues are disclosed.

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THIXOTROPIC CHARACTERISTICS OF COMPACTED CLAYS

H. B. Seed* and C. K. Chan**
(Proc. Paper 1427)

SYNOPSIS

Previous investigations of thixotropic effects in saturated clays are summarized. The existence of thixotropic effects in compacted clays and their variation with soil composition is demonstrated. Test data illustrating the magnitude of thixotropic strength increases with time for compacted clays in normal compression tests and under repeated loading conditions is presented and the significant effects of thixotropy on the results of tests conducted using different rates of loading and different frequencies of repeated loading are discussed.

The detrimental effects of disturbance or remolding on the properties of natural deposits of saturated clay have long been recognized by soil engineers. For the majority of such natural deposits, remolding causes a pronounced reduction in strength even though the composition of the soil is unchanged. This characteristic of saturated clays is expressed by the sensitivity:

$$\text{Sensitivity} = \frac{\text{strength of undisturbed material}}{\text{strength of remolded material}}$$

and the descriptive terms shown in Table 1 have been used to classify clays with different sensitivities. It has been observed that heavily preloaded clays are insensitive(1) but the vast majority of natural clay deposits have sensitivities in the range 2 to 8. However, extra-sensitive clays are not uncommon and a number of examples of the quick clays, which become practically fluid on remolding, have been reported. The highest value so far determined appears to be about 150 for a deposit at St. Thuribe, Canada.(2)

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TABLE 1

Sensitivity	Type of Clay
1 - 0	insensitive
1 - 2	low sensitivity
2 - 4	medium sensitivity
4 - 8	sensitive
8 - 16	extra-sensitive
16	quick-clays

For many years the loss of strength accompanying remolding was attributed mainly to a breakdown of a complex structure of the natural clay. However, more recent investigations have shown that for some materials a good proportion of the loss in strength is due to thixotropy and that the sensitivity of some moderately sensitive deposits may possibly be attributed entirely to thixotropy.

Thixotropy

As originally defined by Freundlich,(3) thixotropy is an iso-thermal reversible sol-gel transformation. It has been more recently described(4) as a process of softening caused by manipulation or working followed by a gradual return to the original strength when the material is allowed to rest. Since the process of softening and stiffening is completely reversible in a thixotropic material, the phenomenon in soils excludes any changes in water content or chemical composition of the soil.

The properties of a purely thixotropic material have been illustrated by Skempton and Northey(1) as shown in Fig. 1. In its undisturbed state the material has a shear strength of value c. When tested at the same rate of shear immediately after remolding the shear strength is reduced to a value c_r. If the material is then allowed to remain under constant external conditions and without any change in composition, the strength will gradually increase and after a sufficient length of time the original strength c will be regained.

Fig. 2 shows the thixotropic strength increase for three clay minerals as measured by Skempton and Northey, who report, "Kaolin shows almost no thixotropy and illite shows only a small effect. In contrast, the bentonite shows a remarkable regain at very short time intervals and it is not possible to suggest an upper limit for this material since the strength continued to increase throughout the experiment." It is interesting to compare these results with the increase in strength due to thixotropy in natural clays shown in Fig. 5. It is apparent that thixotropy is influenced by other factors in addition to the clay minerals present in the soil.

Thixotropy in Saturated Clays

Several investigations have been conducted to determine the extent to which the sensitivity of natural deposits of saturated clays may be attributed to thixotropy. Moretto(5) conducted tests to measure the strength increases with time of four soils maintained at various constant water contents.

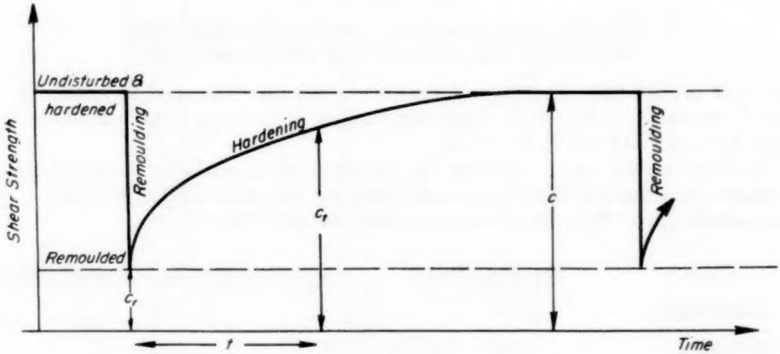
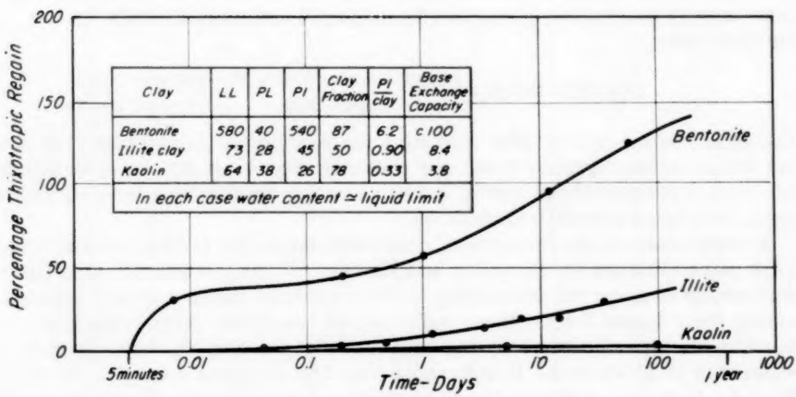


Fig. 1-STRENGTH REGAIN IN A THIXOTROPIC MATERIAL



Clay	LL	PL	PI	Clay Fraction	$\frac{PI}{clay}$	Base Exchange Capacity
Bentonite	580	40	540	87	6.2	c 100
Illite clay	73	28	45	50	0.90	8.4
Kaolin	64	38	26	78	0.33	3.8
In each case water content ≃ liquid limit						

Fig. 2 - THIXOTROPIC REGAIN IN THREE CLAY MINERALS.

(After A.W. Skempton and R.D. Northey)

Typical results of tests on a Laurentian clay showed that this material, when tested immediately after remolding at a water content approximately equal to its liquid limit, had a strength of only about 26 gm. per sq. cm., but the strength increased to about 114 gm. per sq. cm. after 610 days, by which time it had acquired a sensitivity of about 4.4. The acquired sensitivity is defined as the ratio:

$$\frac{\text{strength of clay tested at time t after remolding}}{\text{strength of clay immediately after remolding}}$$

The rate of increase in acquired sensitivity for this material is shown in Fig. 3, from which it will be seen that further increases in sensitivity even after 610 days are likely to occur.

Similar results were obtained for the other three soils investigated but the strength increase for these materials was not so great as that for the Laurentian clay. Typical results are summarized below:

Clay	Liquidity Index	Acquired Sensitivity at 60 days
Laurentian	0.99	3.2
Detroit I	0.98	2.0
Detroit II	0.94	1.6
Mexico City	1.03	1.4

The investigation also showed that the acquired sensitivity decreases with a decrease in water content of the clays. The water content of a soil may be conveniently expressed in relation to its liquid and plastic limits by the liquidity index:

$$\text{Liquidity Index} = \frac{\text{natural water content} - \text{plastic limit}}{\text{liquid limit} - \text{plastic limit}}$$

The acquired sensitivity after 100 days for samples of Laurentian clay at various values of the liquidity index are shown in Fig. 4. It will be seen that after this time a sample of the soil at a water content approaching the plastic limit appears to be practically insensitive.

Further data on the thixotropic characteristics of a number of saturated clays were obtained by Skempton and Northey.(1) Fig. 5 summarizes the rate of increase in acquired sensitivity for five clays at water contents approximating their liquid limits; these data include two of the clays tested by Moretto. In a period of about one year three of the clays had acquired a sensitivity of 2, while the Beauharnois clay had acquired a sensitivity of about 4. However, although these materials, especially the Beauharnois clay, acquired considerable strength as a result of thixotropy, the strength acquired in one year was, for three of the clays, only a small proportion of the original strength loss which occurred upon remolding.

In comparing the natural sensitivity of a clay with that acquired due to thixotropy, it is necessary to consider the age of the undisturbed soil and the probable increase in thixotropic strength throughout the entire period since its deposition. Comparisons of the rate of strength increase due to thixotropy with the loss in strength caused by remolding were made for six soils by Skempton and Northey. The results are shown in Figs. 6 and 7.

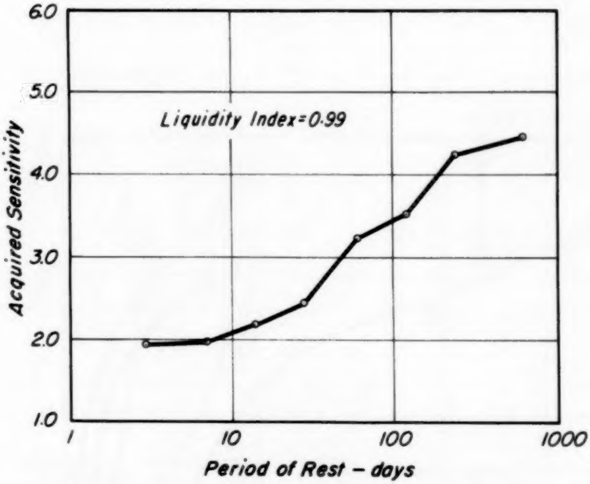


Fig. 3-EFFECT OF PERIOD OF REST ON STRENGTH OF LAURENTIAN CLAY.

(After O. Moretto)

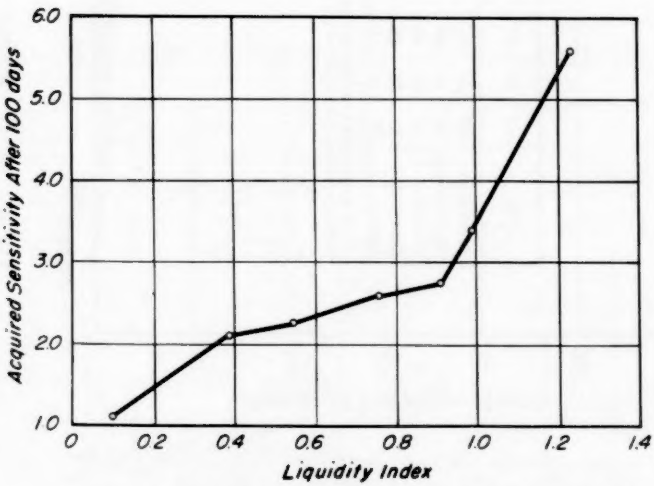
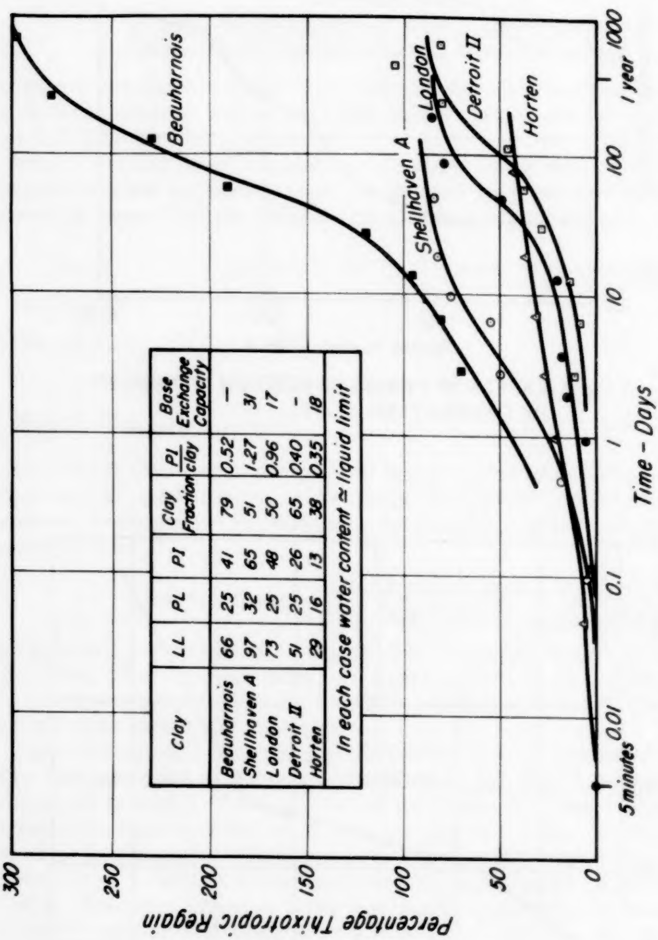


Fig. 4-VARIATION OF ACQUIRED SENSITIVITY WITH LIQUIDITY INDEX FOR LAURENTIAN CLAY.

(After O. Moretto)



Clay	LL	PL	PI	Clay Fraction	PI/clay	Base Exchange Capacity
Beauharnois	66	25	41	79	0.52	—
Shellhaven A	97	32	65	51	1.27	31
London	73	25	48	50	0.96	17
Detroit II	51	25	26	65	0.40	–
Horten	29	16	13	38	0.35	18

In each case water content ≃ liquid limit

Fig. 5 - THIXOTROPIC REGAIN IN SOME TYPICAL CLAYS.

(After A.W. Skempton and R.D. Northey)

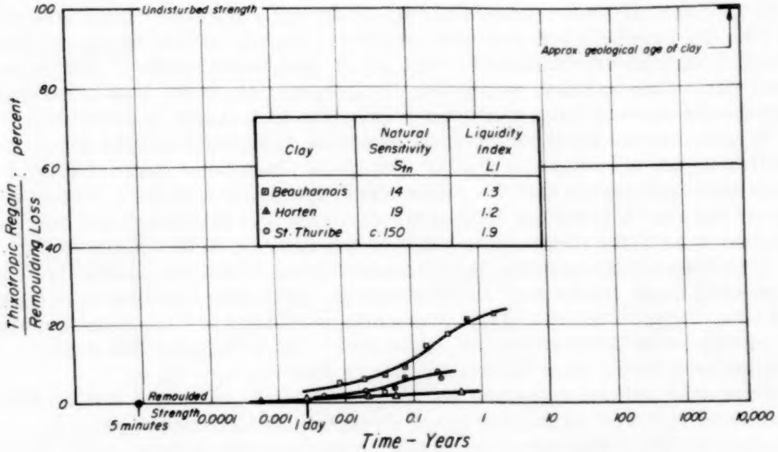


Fig.6–THIXOTROPIC REGAIN IN EXTRA SENSITIVE CLAYS.

(After A.W. Skempton and R.D. Northey)

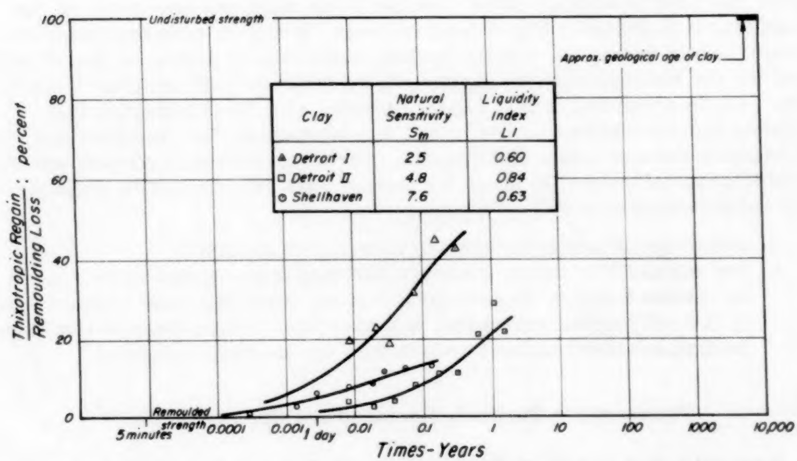


Fig. 7 –THIXOTROPIC REGAIN IN CLAYS OF MODERATE SENSITIVITY.

(After A.W. Skempton and R.D. Northey)

The proportion of the original strength loss which has been regained at different times after remolding is shown in Fig. 6 for three clays of high sensitivity. The estimated age of these clays is about 5,000 to 10,000 years. It will be seen from the measured rate of strength increase that in this period of time it is unlikely that the clays would regain, due to thixotropy, the full strength loss which occurred on remolding. Although thixotropy can account for a part of the original sensitivity, the greater part of the loss in strength that occurs on remolding would seem to be due to a change in structure.

Similar curves for three clays of moderate sensitivity and the same geological age are shown in Fig. 7. For these clays there appears to be a reasonable possibility that the entire strength loss on remolding might be recovered over a period of 5000 years as a result of thixotropy and the original sensitivity might conceivably be due entirely to this phenomenon.

This does not necessarily mean that thixotropy alone can account for the sensitivity of all moderately sensitive clays. Test data reported by Berger and Gnaedinger[6] for two clays of medium sensitivity and the same geological age as those discussed above, show that it is improbable that their original sensitivity could be entirely due to thixotropy.

Thus most natural clays are not truly thixotropic materials and are likely to recover only part of their original strength loss on remolding due to thixotropy. The influence of remolding on the strength of these materials is thus represented by Fig. 8. However, even a partial recovery of the original loss in strength may on occasions have important practical applications.

The difference in thixotropic behavior of different clays might be attributed in part to differences in their liquidity indices. Both Moretto and Skempton and Northey observed that thixotropic strength regain decreases with decreasing water content below the liquid limit. The ratio of the acquired sensitivity after 100 days at various water contents to the acquired sensitivity at the liquid limit is shown in Fig. 9 for five clays. It will be seen that the thixotropic regain decreases with decreasing water content below the liquid limit, and for the Beauharnois clay appears to approach zero at the plastic limit. The results presented in this figure, together with the observation that heavily preconsolidated boulder clays are insensitive, led Skempton and Northey to the conclusion that "heavily over-consolidated clays with water contents typically equal to about the plastic limit are insensitive probably for the following reasons:

a. thixotropy is negligible at such water contents, and
b. any meta-stable micro structure that may have existed in the clay at an earlier stage in its geological history, when still under comparatively low overburden pressures, will have been broken down by the intense loading and deformation to which the clay has been subjected."

Thixotropy in Partially Saturated and Compacted Clays

Very little data has been reported to show the influence of thixotropy in partially saturated or compacted clays. Hirashima[7] has described difficulties in compacting a volcanic ash with an extremely high water content during the construction of embankments in Hawaii.

"Due to its naturally high moisture content and its thixotropic nature, the ash is an extremely difficult material to compact. This is because the process of spreading and compacting necessarily manipulates the material so

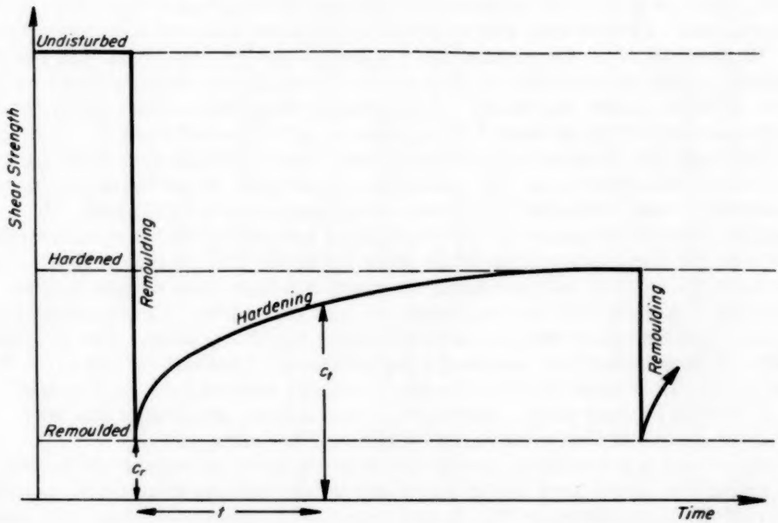


Fig. 8-STRENGTH REGAIN IN A PARTIALLY THIXOTROPIC MATERIAL

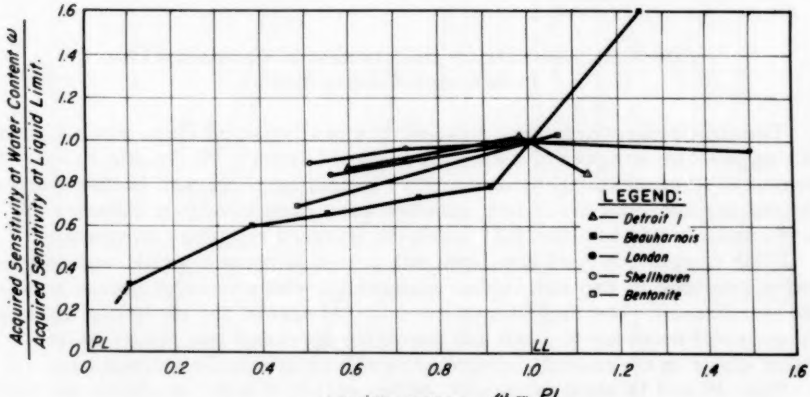


Fig. 9 - EFFECT OF WATER CONTENT ON THIXOTROPIC REGAIN.

(After A.W. Skempton and R.D. Northey)

that the ash, which in cut sections is in a solid state, is worked into a plastic state. Upon further working, the plastic state is changed to the semi-liquid state, all without additional moisture. The usual types of compacting equipment, such as flat wheel, sheeps-foot and pneumatic tired rollers have proved impractical. Compaction with bulldozers has given the best results thus far.

"In general, it is not possible to compact in thin layers. With thin layers a given volume of material is subject to a more intense working than in the case of thick layers, the weight of compacting equipment remaining the same. A thickness of layer of about 3 ft. appears to give the best results. With such a thick layer the compacting equipment does not impart plasticity throughout the entire thickness of the lift. Hence the upper part of the lift is compacted in a plastic state while the lower part is compacted in a solid state. If bogging down of equipment is imminent, it is imperative to cease operations and transfer the equipment to some other location. During this period of rest the material will not only gradually compact due to its own weight, but the part that is plastic will set and regain its lost consistency. This setting is not due to dehydration but is a result of jellifying of the mass. The embankment can then be further compacted and additional material spread on it. The above process is slow but it is the most reliable method thus far evolved."

In view of its high water content, it is perhaps not surprising that this material should exhibit pronounced thixotropic characteristics.

Most clays are normally compacted at water contents nearer the plastic limit than the liquid limit and at these low water contents it would be expected, from the data presented in Fig. 9, that thixotropic effects would be very small. Furthermore, tests conducted by Leonards(8) on a compacted clay showed no tendency for this material to increase in strength over a period of about a year. However, the analysis of test results during an investigation of soil deformation under repeated loading led to the belief that this is not always the case. As a result the following studies of thixotropic effects on compacted clays were conducted.

Effect of Remolding on Deformation of Compacted Clay in Repeated Loading Tests

The first evidence of thixotropic effects in compacted clays was obtained during tests on samples of a silty clay (Liquid Limit = 37, Plastic Limit = 23) subjected to repeated applications of a constant axial stress. In these, and all the subsequent tests described, samples were compacted to a diameter of 1.4", and a height of about 3.5" using the Harvard miniature compactor.

After compaction each specimen was placed between a lucite cap and base and surrounded by two thin rubber membranes with a layer of grease between the membranes. The membranes were sealed against the lucite cap and base by means of neoprene 0-rings and the entire specimen was placed under water either in the triaxial compression cell or in a lucite storage tank.

Figs. 10 and 11 show the results of two series of tests on specimens having a water content of 18 percent, a dry density of 112 lb. per cu. ft. and a degree of saturation of 95%. These specimens were placed in triaxial compression cells as for a normal type of unconsolidated-undrained test, a confining pressure of 1 kg. per sq. cm. was applied but instead of gradually increasing the deviator stress to failure, a deviator stress of 0.8 kg. per sq. cm. was repeatedly applied and removed, each application having a duration of 0.2 seconds, with a frequency of 20 applications per minute.

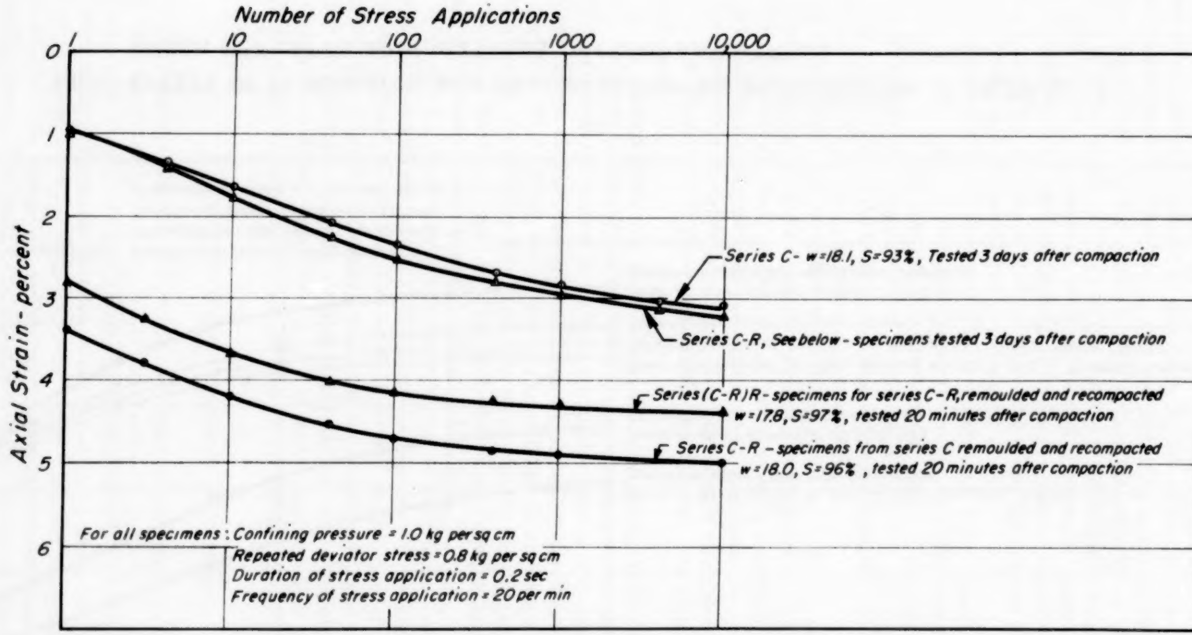


Fig. 10-EFFECT OF STORAGE AND REMOULDING AT CONSTANT WATER CONTENT ON DEFORMATION OF SILTY CLAY UNDER REPEATED APPLICATIONS OF CONSTANT STRESS

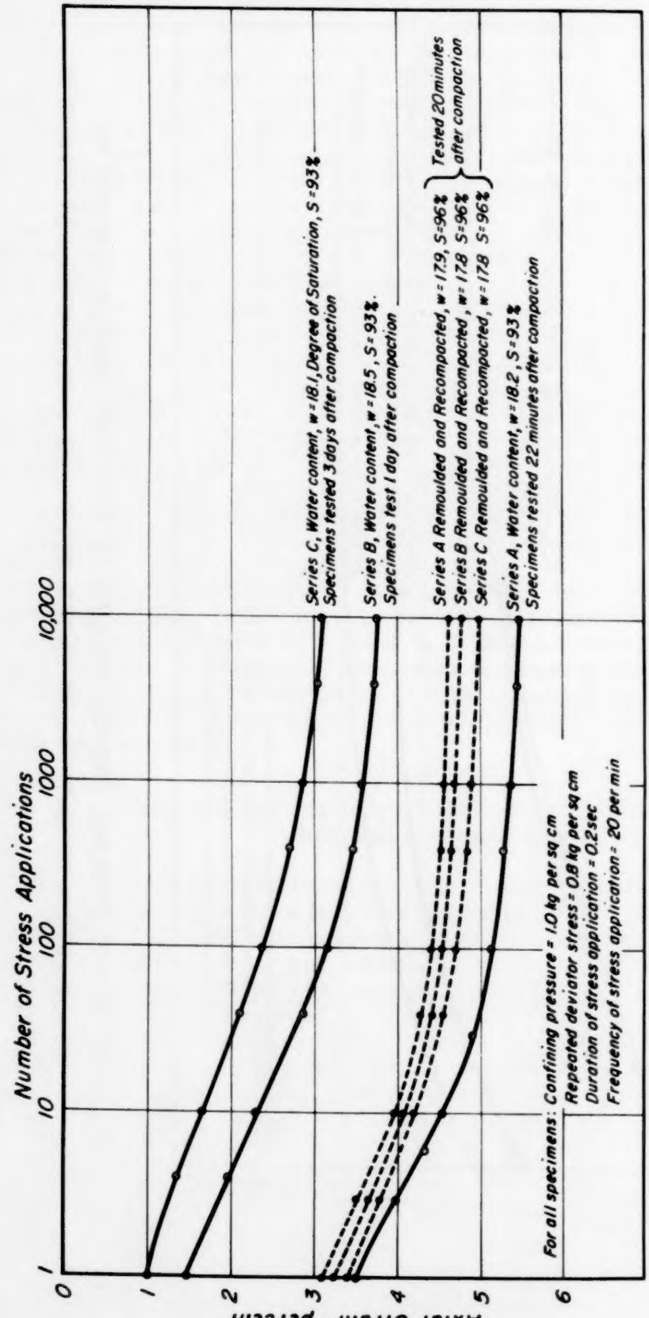


Fig. 11—EFFECT OF REMOULDING AND RECOMPACTION ON DEFORMATION OF SILTY CLAY UNDER REPEATED APPLICATIONS OF CONSTANT STRESS.

In the first series of tests, several specimens were tested three days after compaction. The average deformation of these specimens under various numbers of stress applications is shown by the curve for Series C in Fig. 10. The specimens were then broken up and recompacted in the same manner as before. As a result of the slight drying during this operation, the recompacted specimens had slightly lower water contents and slightly higher densities. Several of these specimens were subjected to repeated loading 20 minutes after recompaction and their deformations are shown by the curve for Series C-R in Fig. 10. It will be seen that these specimens deformed about twice as much as those tested previously.

Several others of the recompacted specimens were stored for three days at which time their average deformation under repeated loading was almost identical with that of the original specimens. Again these specimens were broken up and recompacted (Series (C-R)R in Fig. 10) and when tested 20 minutes after compaction showed a considerable increase in deformation characteristics, as may be seen in the figure.

It would appear from these tests that a period of storage without any appreciable change in water content causes a significant increase in the resistance to deformation of compacted specimens in repeated loading tests. Furthermore, this increased resistance to deformation can be destroyed by recompaction of the specimens and then once again regained on further storage. The effect is therefore reversible and has the characteristics of thixotropy.

In the second series of tests compacted specimens were tested 20 minutes after compaction, one day after compaction and three days after compaction with a progressive decrease in the deformation occurring under the same series of repeated stress applications, as shown by the curves for Series A, B, and C in Fig. 11. In each case the specimens were broken up and recompacted after being subjected to 10,000 stress applications and then retested 20 minutes after compaction. The deformations of the recompacted specimens are shown by the dashed curves in Fig. 11.

Although the deformations of the original specimens were markedly different, the deformations of the recompacted specimens were almost identical. It would appear, therefore, that remolding and recompaction can completely destroy the strength increases resulting from different periods of storage and result in specimens having similar deformation characteristics.

It is believed that these two series of tests provide clear evidence of the existence of thixotropic characteristics in compacted clays.

Effect of Water Content on Thixotropy

In the test series just described, the specimens were compacted on the wet side of optimum to a degree of saturation of about 95%. In order to determine if the thixotropic effects observed were a result of the high degree of saturation of these specimens, a series of tests were conducted on samples of the same soil prepared at different water contents using the same compactive effort. The stress vs. strain relationships of these samples were determined by normal triaxial compression tests of the unconsolidated-undrained type using a lateral pressure of 1 kg. per sq. cm.

Samples were prepared at four different water contents. At any one water content four samples were compacted, two of which were tested immediately

after compaction and two tested after a one-week period of storage. The water contents and densities for all specimens are shown in Fig. 12. It will be seen that all the specimens lie essentially on the same compaction curve.

Typical stress vs. strain curves for samples having high and low degrees of saturation are shown in Fig. 13 (a) and (b). It will be seen that in each case the specimens tested after a period of storage had a higher strength than those tested immediately after compaction, though the difference appears to be greater for the specimens having a high degree of saturation. This result is shown more clearly in Fig. 12 which shows the stress required to cause 10% strain for all the specimens tested. It is apparent that a period of storage causes an increase in strength for specimens at all water contents.

In order to compare the magnitude of the thixotropic strength increase, the term "thixotropic strength ratio" was introduced. This is defined as follows:

$$\text{Thixotropic strength ratio} = \frac{\text{strength of specimen tested after time t}}{\begin{array}{c}\text{strength of identical specimen tested}\\ \text{immediately after compaction}\end{array}}$$

This term for partially saturated specimens corresponds to the acquired sensitivity of saturated specimens. It is necessary to adopt this term, however, because the strength of partially saturated soils is not independent of the confining pressure and different values of thixotropic strength ratio may be obtained depending on the magnitude of the lateral pressure used in the triaxial compression tests.

For the above tests, the thixotropic strength ratios at different water contents can be determined by a comparison of the ordinates of the two strength vs. water content curves and the computed values are plotted in Fig. 12. It will be seen that there is a pronounced increase in thixotropic effects at a water content of about 17% which is very close to the optimum water content for the compactive effort used. It would appear, therefore, that thixotropy in compacted soils is of special significance at higher degrees of saturation and is relatively small at low degrees of saturation. However, it should be remembered that the thixotropic strength ratios shown in Fig. 11 were determined after only 1 week of storage and might be considerably greater if the samples were allowed to rest for longer periods.

In the design of pavements the permissible strain which a soil may develop before failure is considered to occur is considerably less than 10% and may be more of the order to 5%. The thixotropic strength ratios when failure is defined as the stress required to cause 5% strain are shown in Fig. 14; it will be seen that they are appreciably greater than those determined for higher strains and also become more significant at lower values of water content.

A summary of thixotropic strength ratios determined at different values of axial strain is shown in Fig. 14. It will be seen from this figure:

1. that thixotropic effects become increasingly significant at smaller strains,
2. that thixotropic effects are relatively small for samples compacted on the dry side of optimum for the compactive effort being used, and
3. that thixotropic effects even after 1 week may be quite appreciable for samples compacted on the wet side of optimum.

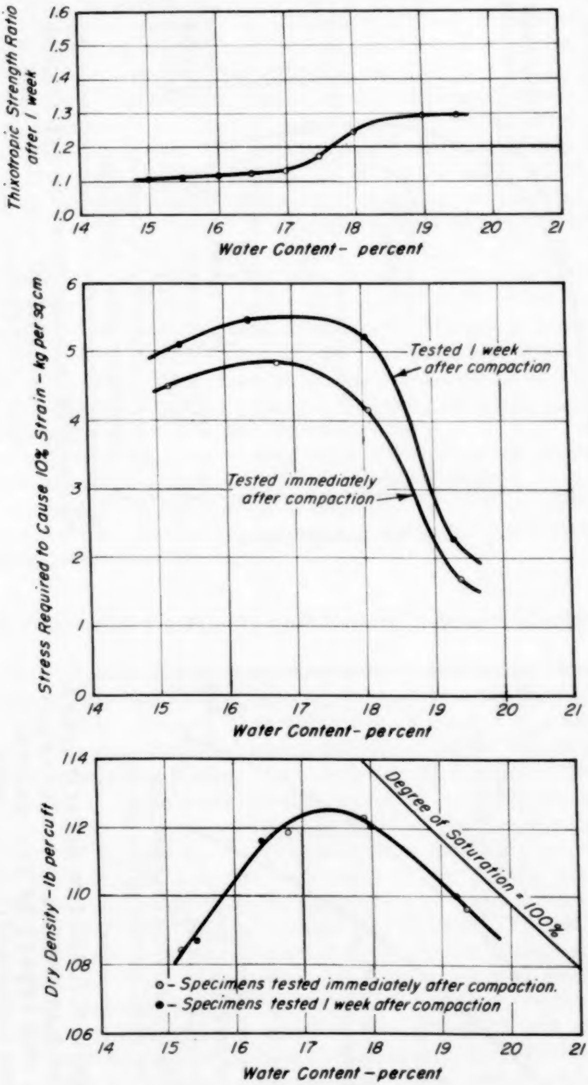


Fig. 12 - COMPOSITION, STRENGTHS AND THIXOTROPIC STRENGTH RATIOS FOR SPECIMENS OF SILTY CLAY TESTED IMMEDIATELY AND 1 WEEK AFTER COMPACTION.

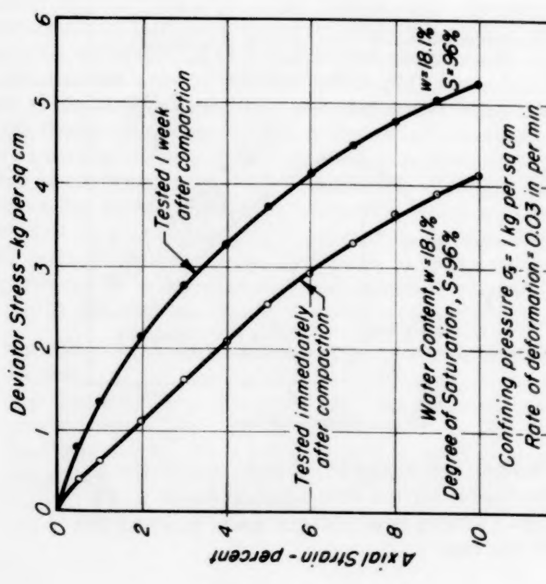


Fig. 13a–STRESS vs DEFORMATION RELATIONSHIPS FOR SIMILAR SPECIMENS OF SILTY CLAY WITH HIGH DEGREE OF SATURATION TESTED IMMEDIATELY AND 1 WEEK AFTER COMPACTION.

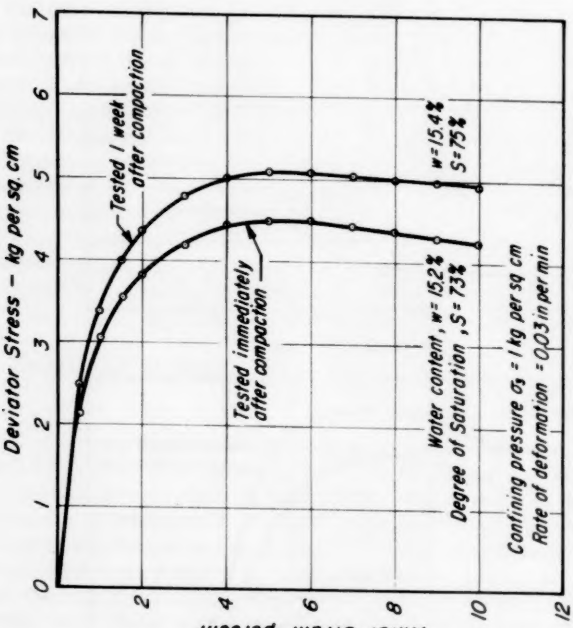


Fig. 13b– STRESS vs DEFORMATION RELATIONSHIPS FOR SIMILAR SPECIMENS OF SILTY CLAY WITH LOW DEGREE OF SATURATION TESTED IMMEDIATELY AND 1 WEEK AFTER COMPACTION.

Increase in Thixotropic Effects with Time in Normal Strength Tests

The increase in thixotropic strength with time for compacted samples of the same silty clay soil is shown by the data presented in Fig. 15. The stress vs. strain curves in this figure were obtained by normal triaxial compression tests of the unconsolidated-undrained type on specimens compacted on the wet side of optimum and tested after different periods of storage. The general pattern of the curves is somewhat similar to that obtained for saturated clays.

If failure is considered to occur at a strain of 10%, the increase in strength with time is shown in Fig. 16. The corresponding increase of thixotropic strength ratio with time is shown in Fig. 17. It will be seen that this soil (Vicksburg silty clay) developed a thixotropic strength ratio of about 1.5 after 28 days and that further increases in strength beyond this period are likely to occur.

Similar results for a highly plastic clay (Liquid Limit = 59 Plastic Limit = 27) and for a sandy clay (Liquid Limit = 35 Plastic Limit = 20) are shown in Fig. 17. It will be seen that for these soils the thixotropic effects are considerably smaller than for the silty clay, and, for the highly plastic clay, little further increase in thixotropic strength is likely to occur after a period of 28 days. This data provides further evidence that the magnitude of thixotropic effects is not related directly to the Atterberg limits.

Consideration has been given to the fact that the observed increase in strength with time might possibly be due to a redistribution of water in the specimens resulting from the method of storage. However, a careful study of the water content at the outside and inside sections of specimens has failed to reveal any measurable difference.

Increase in Thixotropic Effects with Time in Repeated Loading Tests

The possible significance of thixotropic effects in tests on compacted samples subjected to repeated loading is readily seen from the data presented in Fig. 18. In this series of tests 18 samples were prepared at a water content of 18.4% and compacted to a degree of saturation of 92%. The samples were then subjected to repeated loading tests in groups of three at different periods after compaction. In these tests the samples were placed in triaxial compression cells as for an unconsolidated-undrained type of test using a confining pressure of 1.0 kg. per sq. cm., but instead of gradually increasing the deviator stress to failure, a deviator stress of 0.8 kg. per sq. cm. was repeatedly applied and removed, each application having a duration of 0.2 second, with a frequency of 20 applications per minute. The tests were continued until 10,000 stress applications had been applied to each specimen.

The deformations of the samples resulting from different numbers of stress applications are shown in Fig. 18; the curves in this figure represent the averages for the three specimens tested in each group. It will be seen that a three-day period of storage prior to testing caused a reduction of almost 50% in the deformations of the specimens and that further reductions resulted from longer periods of storage. The large increase in resistance to deformation occurring during the first three or four days after compaction is clearly shown in Fig. 19 (a) and the probability of further stiffening even after 50 days of storage by Fig. 19 (b).

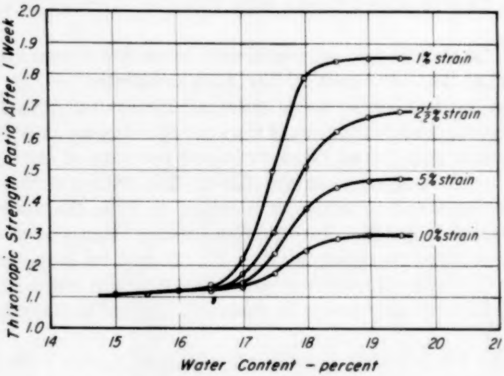


Fig.14 - THIXOTROPIC STRENGTH RATIOS FOR DIFFERENT VALUES OF AXIAL STRAIN.

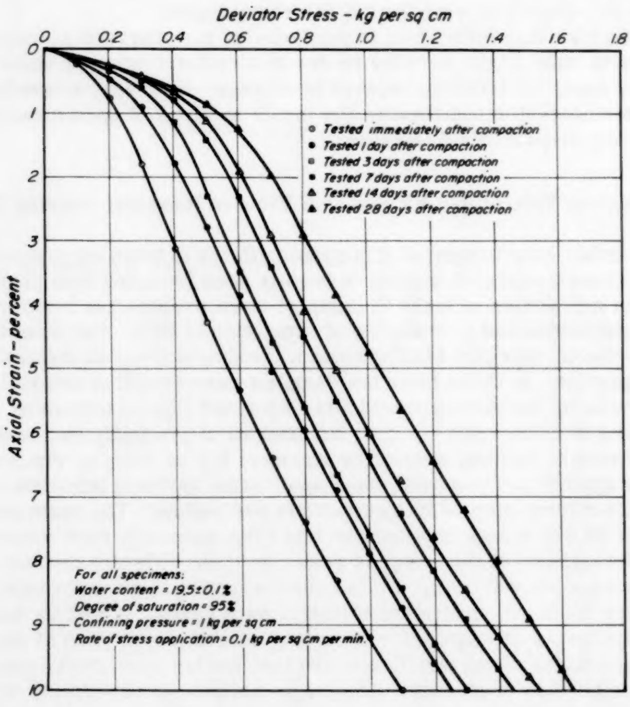


Fig. 15 - EFFECT OF PERIOD OF STORAGE AT CONSTANT WATER CONTENT ON STRESS vs DEFORMATION RELATIONSHIPS FOR SILTY CLAY WITH HIGH DEGREE OF SATURATION.

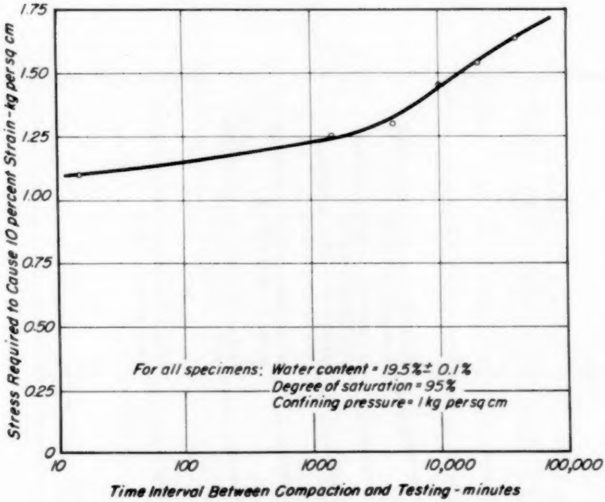


Fig. 16 - EFFECT OF PERIOD OF STORAGE AT CONSTANT WATER CONTENT ON STRENGTH OF SILTY CLAY.

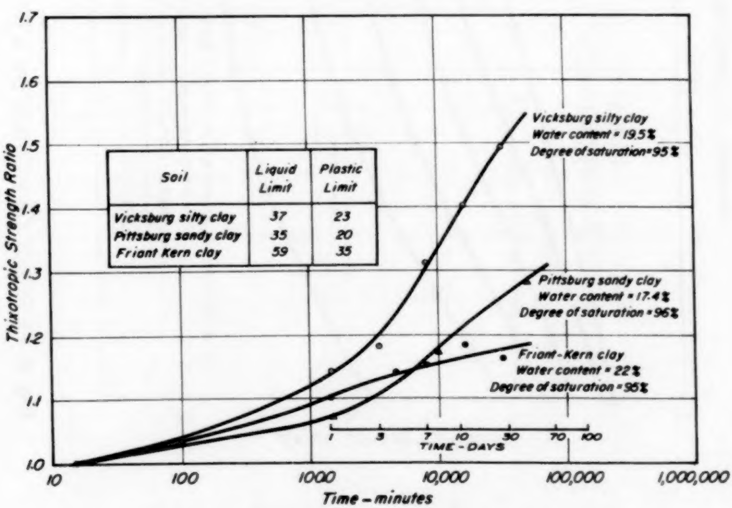


Fig. 17 - INCREASE IN THIXOTROPIC STRENGTH WITH TIME FOR THREE COMPACTED CLAYS.

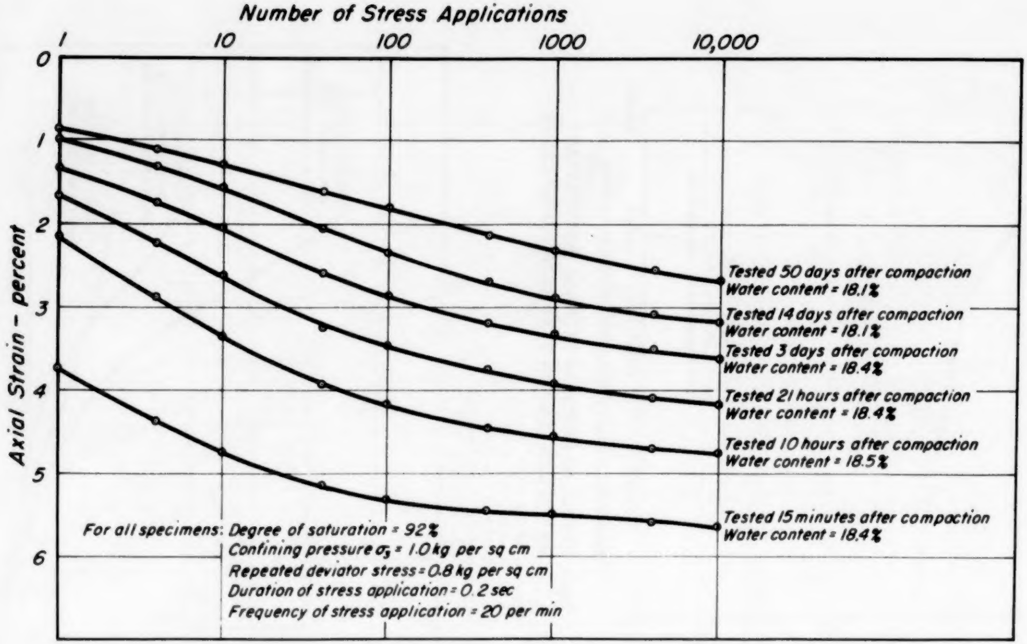
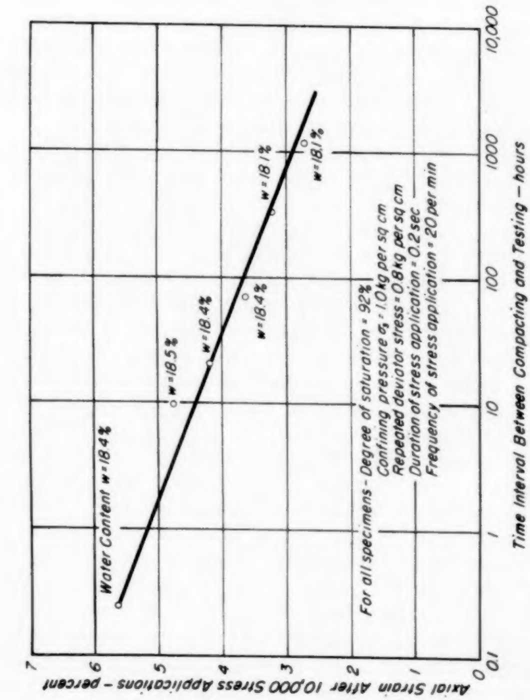
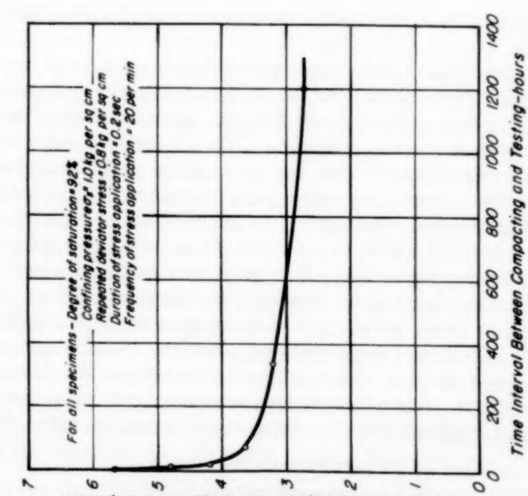


Fig. 18 - EFFECT OF PERIOD OF STORAGE AT CONSTANT WATER CONTENT ON DEFORMATION OF SILTY CLAY UNDER REPEATED APPLICATIONS OF CONSTANT STRESS.





Figs. 19a (left) and 19b (right) - DECREASE IN DEFORMATION OF SILTY CLAY UNDER REPEATED LOADING RESULTING FROM PERIOD OF STORAGE AT CONSTANT WATER CONTENT.

It is interesting to note that thixotropic stiffening appears to have greater effects in these tests than in the normal type of strength tests. After three days the same soil had a thixotropic strength ratio of only 1.2 in a normal strength test, yet its deformation in repeated loading tests was reduced about 50%. This discrepancy is not surprising, however, in view of the fact that thixotropic effects become increasingly significant at smaller strains and the deformation in the repeated load tests did not exceed about 6%.

The change in deformation characteristics which may result from a period of rest after compaction is indicative of the great difficulty encountered in predicting the probable life of pavements from laboratory tests. In some pavement design procedures failure of the subgrade is considered to occur when the deformation exceeds about 5%. Using a triaxial compression test procedure to represent the stress conditions on an element of soil under a pavement and postulating vehicular traffic of uniform size, weight, speed and frequency, enormous differences in the predicted number of wheel loads or stress applications required to cause failure could result simply from the different rest periods which might be allowed between sample compaction and the start of testing. Thus from the data in Fig. 18, if specimens were tested in the laboratory immediately after compaction, the soil should reach the failure strain of 5% after only about 20 applications; on the other hand, if the specimen were allowed to rest for 1 day before testing, the number of applications required to cause failure would be increased to about 1,000,000, while if the interval between compaction and testing were several days or more, failure might never occur under conceivable numbers of wheel loads.

It is not meant to imply that the probable life of a pavement can be predicted in such a simple fashion from the results of this type of test, but the general nature of the effect and its possible significance are illustrated by these comparisons. The possible error in the predicted life of a pavement which might result from calculations based on tests conducted soon after sample compaction is also apparent.

Effect of Rate of Loading on the Strength of Compacted Clay

The thixotropic characteristics of a compacted clay may have important effects in investigations of the influence of rate of loading on the strength of such a soil. In general, it has been found that for saturated clays the strength is reduced with a decrease in rate of loading. On the other hand, data reported by Casagrande and Wilson(9) for two compacted soils show that the strength is increased by a reduction in the rate of loading. This latter finding may well be due to thixotropy, and in order to throw further light on the result, two series of tests were carried out on the same soil, the first on samples loaded at different rates immediately after compaction and the second on samples allowed to rest for a period of two weeks prior to loading.

A total of 20 samples were compacted at a water content of 18.3% to a dry density of 112 lb. per cu. ft. corresponding to a degree of saturation of about 96%. Five pairs of these samples were subjected to unconsolidated-undrained triaxial compression tests, using a confining pressure of 1 kg. per sq. cm. immediately after compaction using the following rates of loading:

1) 2 kg. load increments at 30 second intervals
2) 2 kg. load increments at 1 minute intervals

3) 2 kg. load increments at 10 minute intervals
4) 2 kg. load increments at 1 hour intervals
5) 2 kg. load increments at intervals of one day

The other ten specimens were stored in rubber membranes under water for two weeks and then tested in pairs using the same test procedure and essentially the same rates of loading.

The results of these tests are presented in Fig. 20 which shows the average strength of the pairs of specimens (determined at the stress required to cause 10% strain) plotted against the time of loading, that is the time required for the specimens to reach 10% strain.

For the specimens tested immediately after compaction, there was a slight but gradual decrease in strength for times of loading between 5 minutes and 100 minutes, but a progressive increase in strength for times of loading greater than 100 minutes. For a test lasting ten days the strength was about 30% greater than that measured in a standard 10 minute test.

For the specimens tested two weeks after compaction increases in time of loading up to one day caused a small decrease in strength, and there was only a very slight increase in strength when the time of loading was increased to 10 days; even so the strength of a specimen in a test lasting for ten days was about 4% less than that in a test lasting ten minutes.

These results would seem to indicate that the pronounced increase in strength at long times of loading for specimens tested immediately after compaction is due primarily to the normal thixotropic increase in strength with time of the specimens. For the specimens tested after being stored for 2 weeks the greater part of the thixotropic strength increase had occurred before the tests were started and the relatively small thixotropic effects during the following ten days for which the longest test was conducted had only a very slight influence on the results.

Further evidence that the influence of rate of loading on the strength of compacted samples tested immediately after compaction depends largely on the thixotropic characteristics of the samples is provided by the results presented in Fig. 21. These data were obtained in tests on samples of the same silty clay soil and show the effect of rate of loading, for tests started immediately after compaction, on the strength of samples having a low degree of saturation. It will be noted that the general form of the curve is similar to that for samples having a high degree of saturation and shown in Fig. 21. However, the magnitude of the strength increase for long times of loading is very much reduced, as will be seen from the comparison of the results in Fig. 22. Since thixotropic characteristics decrease considerably for degrees of saturation below about 85% (see Fig. 12), this result would be expected.

It would appear from these data that, in general, if two specimens have the same properties and these properties do not change with time or deformation, the specimen subjected to the slower rate of loading will tend to have the lowest strength because of the greater amount of creep movement which can take place in the longer intervals between load increments. However, for specimens whose strength is strongly influenced by thixotropy, the longer the duration of the test, the greater will be the thixotropic strength acquired by the specimen. The effect of an increase in time of loading will thus depend, perhaps among other things, on the following two factors:

1. A tendency for the strength to decrease because of the increased time available for creep deformation.

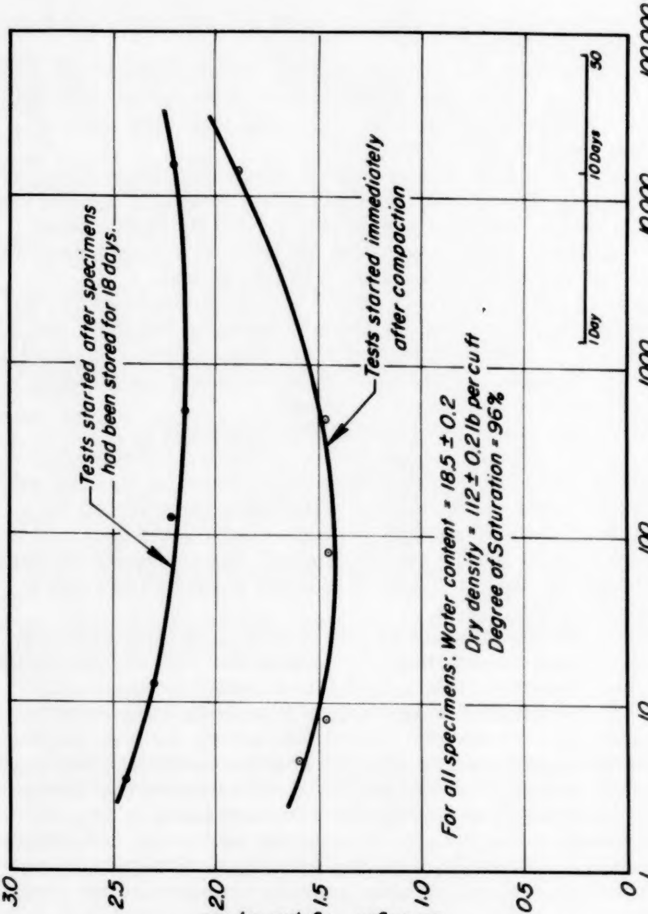


Fig.20-EFFECT OF RATE OF LOADING ON STRENGTH OF SILTY CLAY COMPACTED TO HIGH DEGREE OF SATURATION.

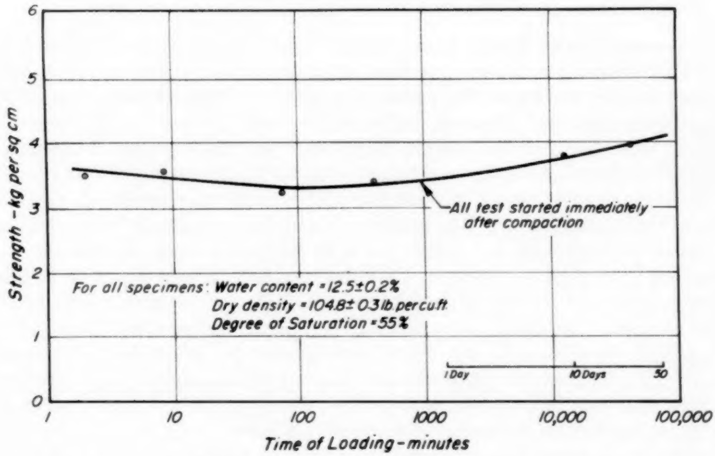


Fig.21- EFFECT OF RATE OF LOADING ON STRENGTH OF SILTY CLAY COMPACTED TO LOW DEGREE OF SATURATION.

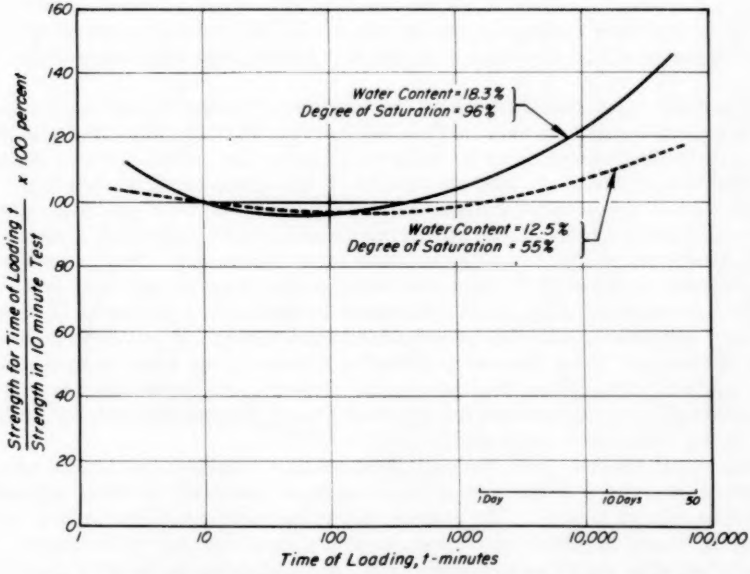


Fig.22- COMPARISON OF EFFECTS OF RATE OF LOADING ON STRENGTH FOR SAMPLES OF SILTY CLAY AT HIGH AND LOW DEGREES OF SATURATION.

2. A tendency for the strength to increase because of the increased creased time available for thixotropic effects to develop.

For the specimens tested immediately after compaction, it could thus be argued that increased creep movements outweigh the influence of thixotropic hardening during the first 100 minutes time of loading, but for longer times of loading thixotropic stiffening is sufficiently great to cause an increase in strength. However, for the specimens tested two weeks after compaction only a small thixotropic strength increase is likely to occur in the interval from 14 to 24 days, during which the longest test was being conducted, see Fig. 17, and thus the increased creep movement will tend to control the result leading to the observed decreases in strength with increased time of loading.

Although this simple concept of the phenomenon would seem to be adequate to explain the observed results, other factors are also likely to influence the behavior of specimens in long time tests under slow rates of loading. In a material which has no structural sensitivity for example, yet has some sensitivity due to thixotropy, tests on undisturbed samples might give results similar to those on newly compacted clay, even though the material exhibits no further increase in thixotropic strength with time. Deformations of such a soil will cause some disturbance and a loss of thixotropic strength. If the total time of the test is insufficient for the thixotropic strength to be recovered, then the strength will decrease with an increase in time of loading. If, however, the rate of loading is so slow that the soil can regain some strength between load increments, the strength might begin to increase with increased time of loading. Thus the relationship between strength and time of loading might be similar to the curves in Fig. 22, though the upward trend in the curve would be expected to occur at a longer time of loading than those shown.

That a soil may gain strength in the intervals between stress applications is illustrated by the test data in Figs. 23 and 24. Fig. 23 shows the results of triaxial compression tests on samples of silty clay compacted to a degree of saturation of 96% at a water content of 19.1%. Each curve in this figure is the average of three tests on identical specimens. The first group of specimens was tested 28 days after compaction and loaded slowly over a period of 16 days with intervals of one day between load increments. The second group of specimens was tested 44 days after compaction (that is, when the tests on the first group were completed) and loaded to failure in a period of 15 minutes with only one minute interval between load increments. It will be seen that these specimens, since they were tested at a longer time after compaction, were initially stiffer than the first group. However, as the deformation increased they became weaker than the first group, despite the fact that they were being loaded at a later date.

This would seem to indicate that the specimens subjected to a slow rate of loading possessed some source of strength not available to those subjected to a rapid rate of loading. The source of this strength could not be due to the normal increase in thixotropic strength with time since they were tested at an earlier date than the others; furthermore, the long interval between load increments permits more creep to occur and would be expected to cause a greater strain under any given applied stress. It seems reasonable to conclude, therefore, that the source of this strength stems primarily from the fact that long time intervals between stress applications enable the specimens to regain some of the thixotropic strength which is lost when they are first

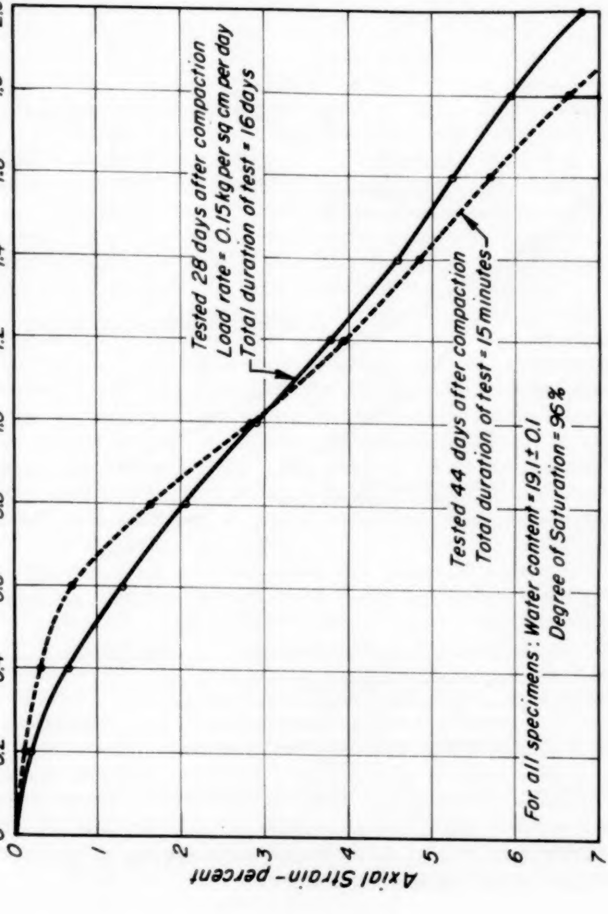


Fig.23 - INCREASE IN RESISTANCE TO DEFORMATION OF SILTY CLAY DUE TO SLOW RATE OF LOADING.

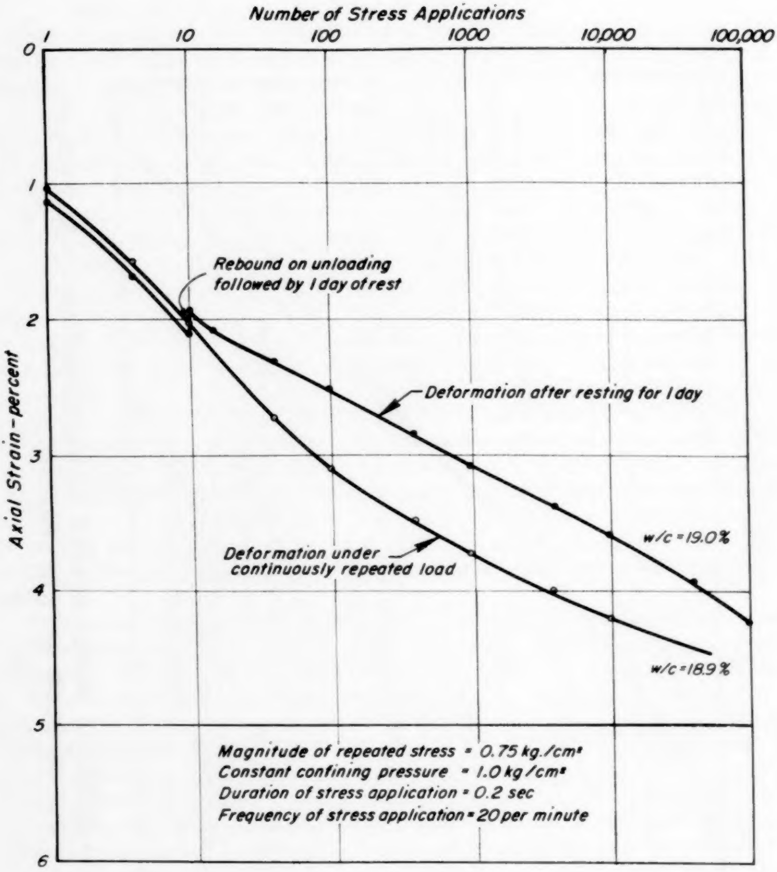


Fig. 24 - EFFECT OF PERIOD OF REST ON DEFORMATION OF SILTY CLAY UNDER REPEATED LOADING.

deformed by the application of a stress increment. On the other hand, with a high rate of loading there is no time between stress increments for such a strength regain to occur and as a result specimens which are initially stronger (see Fig. 22) lose a greater proportion of their thixotropic strength and at higher strains may have lower strengths than specimens subjected to very slow rates of loading.

The strength regain of a compacted soil specimen following deformation is also illustrated by the data in Fig. 24. Here two identical specimens were subjected to repetitions of the same deviator stress at a frequency of 20 applications per minute. On one specimen the applications were continuous until about 10,000 repetitions had been applied. On the other, 10 repetitions of stress were applied and then the specimen was allowed to rest for one day before the applications were continued. It will be seen that after this period of rest the specimen deformed very much less than did the specimen loaded continuously. This difference in deformation characteristics is far greater than would result from the one day difference in age of the specimens at the time the applications were applied (the change in strength characteristics from the 14th to the 15th day is very small) and would appear to result, therefore, from a thixotropic strength regain during the period of rest.

If, as these results would seem to indicate, thixotropic strength may be redeveloped in long intervals between load increments or applications, an increase in strength for very long times of loading might be expected in all types of soils possessing thixotropic characteristics and no sensitivity as a result of natural structure. This would be expected to apply to saturated clays as well as compacted clays. The work of Skempton and Northey has indicated that the sensitivity of some natural deposits of saturated clay may be due entirely to thixotropy. For such materials it would be expected that the effect of rate of loading on strength would be similar to the upper curve in Fig. 20. However, if a soil has considerable sensitivity as the result of natural structure, then the upward trend in the curve for long times of loading is unlikely to occur. Strength loss due to structural disturbance on loading is likely to exceed any strength regain by thixotropy between load increments, with the result that the strength will progressively decrease with an increase in time of loading. Such results for sensitive clays have been reported by Casagrande and Wilson.(9)

The above discussion does not take into account the sudden loss of strength which occurs in some soils at high strains, or possible strength changes resulting from changes in void ratio or grain arrangement which may occur during loading. While these factors might also affect the results, it would appear that thixotropic influences and structural disturbance are likely to be the dominating influence and might reasonably account for the observed effects of rate of loading on the strength of most soils.

Effects of Frequency of Stress Application on Soil Deformation Under Repeated Loading

Since the data in Fig. 24 indicates that there may be a significant strength regain during periods of rest following deformation of a compacted clay possessing appreciable thixotropic characteristics, it would seem logical to expect some influence of frequency of stress application on the deformation of soils possessing thixotropic strength when they are subjected to repeated

loading. A comparison of the deformations of samples of silty clay subjected to repeated applications of a constant stress of the same magnitude and duration but with frequencies of application of 20 per minute and 1 per 2 minutes is shown in Fig. 25. The tests were started one month after compaction of the specimens.

In the upper part of the figure data is shown for samples having a high degree of saturation and thus, one month after compaction, possessing appreciable thixotropic strength. It will be seen that there is considerably more deformation of the specimens for the higher frequency of stress application than for the lower frequency of application. Apparently this is due to the greater thixotropic regain between stress applications for the lower frequency. It should be noted that this effect is due to a regain of the thixotropic strength lost during the deformation occurring under the first few load increments and not to the normal increase in thixotropic strength of the specimens with time. The magnitude of the latter effect after 30 days of storage is negligible compared with that observed in the tests.

It would be expected, from the data previously presented, that for soils in which thixotropic effects are small the influence of frequency of stress application on the deformation occurring under repeated loading would also be small. This is illustrated by the data shown in Fig. 25b for samples of the same silty clay compacted to a low degree of saturation. Thixotropic effects in these samples are very small and, as illustrated in Fig. 25b, variations in frequency have only slight effects in repeated loading tests. It is interesting to note in this connection that G. P. Tschebotarioff and G. W. McAlpine(10) found no influence of frequency of stress application at all in repeated loading tests on sands.

CONCLUSION

In conclusion it is suggested that thixotropy can cause substantial changes in strength in some compacted clays, as it does in saturated clays, and may have significant effects on the strength of these soils tested after a period of storage, under conditions of slow increase in stress or under conditions of repeated stress application. For example, Fig. 26 shows the Mohr stress circles (total stresses) and the envelope of failure for unconsolidated-undrained tests on specimens of a thixotropic soil tested using a normal rate of loading (corresponding to a testing time of about 10 minutes) immediately after compaction. The curved envelope can be approximated by a straight line over the range of pressures investigated and the position of this line expressed by the Coulomb equation:

$$s = c_e + p \tan \phi_e$$

where c_e is the intercept on the vertical axis and ϕ_e the inclination of the failure envelope to the horizontal.

For specimens of the same composition tested two weeks after compaction the test results shown in Fig. 26b were obtained and it will readily be seen from the comparison in Fig. 26c that the position of the equivalent straight-line strength envelope is significantly changed. However, the slope of the line is essentially the same and the change is confined simply to the intercept on the shear stress axis. It would be expected from the nature of thixotropy

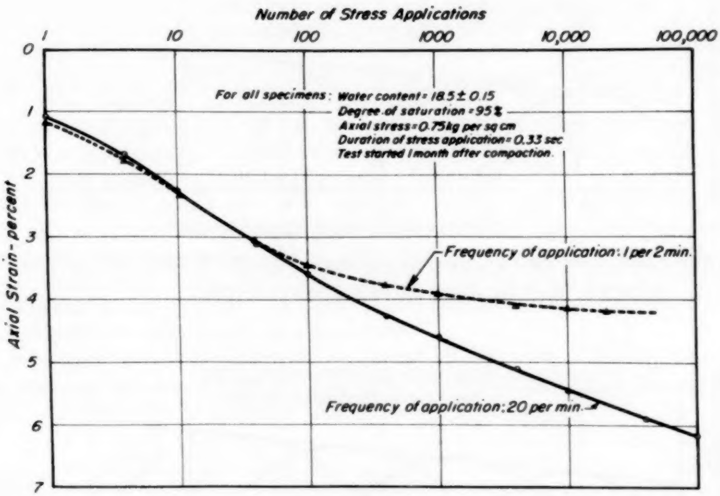


Fig.25a- EFFECT OF FREQUENCY OF REPEATED STRESS APPLICATION ON DEFORMATION OF SILTY CLAY AT HIGH DEGREE OF SATURATION.

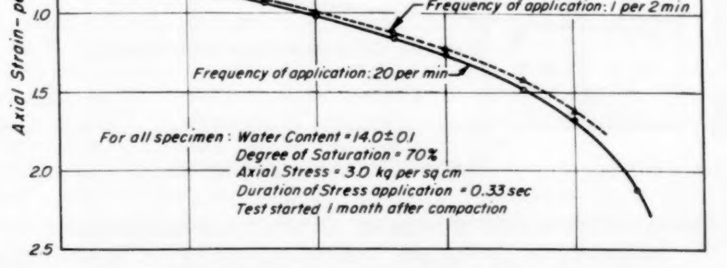


Fig.25b- EFFECT OF FREQUENCY OF REPEATED STRESS APPLICATION ON DEFORMATION OF SILTY CLAY AT LOW DEGREE OF SATURATION.

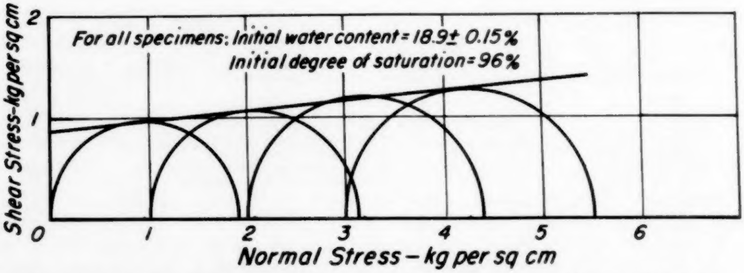


Fig.26a-ENVELOPE OF FAILURE IN UNDRAINED TESTS FOR SPECIMENS TESTED IMMEDIATELY AFTER COMPACTION.

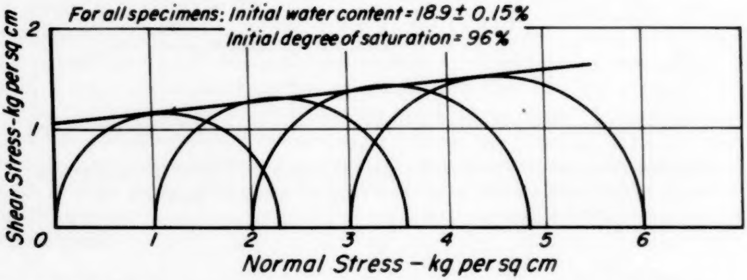


Fig.26b-ENVELOPE OF FAILURE IN UNDRAINED TESTS FOR SPECIMENS TESTED 2 WEEKS AFTER COMPACTION.

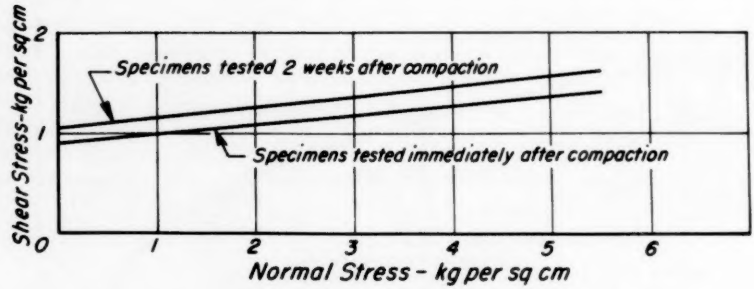


Fig.26c-EFFECT OF PERIOD OF STORAGE AT CONSTANT WATER CONTENT ON ENVELOPE OF FAILURE FOR UNDRAINED TESTS ON SILTY CLAY.

that a change in strength resulting from a period of storage would be independent of the confining pressure and would be reflected by the intercept on the vertical axis of the Mohr diagram. Since, however, the deformation occuring during these tests will destroy some of the original thixotropic strength gain, it would appear that in the standard type of strength test having a duration of about 10 minutes, samples having the same initial composition are likely to lose the same proportion of their original thixotropic strength.

This would not be true, however, for samples tested at the same age with different rates of loading. In this case, specimens tested with a slower rate of loading have a greater thixotropic regain following deformation and ultimately exhibit a higher strength than those tested with a faster rate of loading. This is illustrated by the data in Fig. 27, which shows a comparison of the envelopes of failure for similar specimens tested at the same age but with widely different rates of loading. Again it will be seen that the effect of rate of loading is confined to a vertical displacement of the envelope of failure and is independent of the confining pressure.

Thus it appears that the value of c_e in the approximate Coulomb equation is a function of both the age of the specimen at the start of the tests (t_a) and the time of loading (t_ℓ). It might therefore be expressed

$$c_e = c \cdot f(t_a) \cdot \phi(t_\ell)$$

where c denotes the value of the intercept resulting from causes other than thixotropy, the $f(t_a)$ and $\phi(t_\ell)$ are functions of the age of specimens and time of loading respectively.

It is recognized that the above result was obtained from a comparison of strength envelopes for total stresses in triaxial compression tests and that a more conclusive determination of the effects of thixotropy could have been made if the envelopes for effective stresses had been used. Also that the effects of changes in rate of loading on soil strength may be due to other factors in addition to thixotropy. Nevertheless, the result is apparently applicable to unconsolidated-undrained tests and there would seem to be sufficient evidence that somewhat similar effects might be obtained for envelopes of failure based on effective stresses.

Furthermore, the results would be expected to apply to all types of soil possessing thixotropic strength but no sensitivity due to natural structure. These would include remolded saturated clays after they had developed thixotropic strength following a period of rest and apparently to some natural clays whose sensitivity may be entirely due to thixotropy. A study of the influence of thixotropy on the results of tests conducted at widely different rates of loading might well serve to clarify the meaning of the terms 'true cohesion' and 'true angle of internal friction' for saturated clays.

It would also appear from the results that thixotropic changes are not limited to soils having high liquidity indices but can also occur at water contents near and below the plastic limit. In compacted soils the influence of thixotropy appears to be significant at high degrees of saturation, resulting in some cases in a strength increase of 50% in a period of 4 weeks, but very small for samples compacted on the dry side of optimum for the compactive effort being used. While most soils are compacted on the dry side of optimum, there are nevertheless conditions encountered in practice, both in compaction of clay soils for earth dams and pavement construction, where

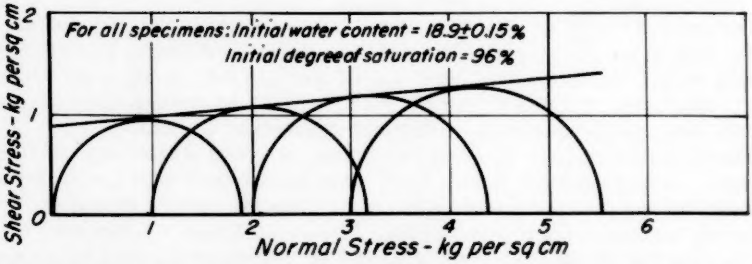


Fig.27a-ENVELOPE OF FAILURE IN UNDRAINED TESTS FOR SPECIMENS TESTED IMMEDIATELY AFTER COMPACTION USING NORMAL RATE OF LOADING.

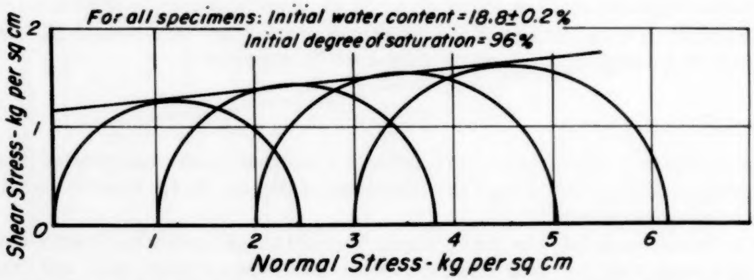


Fig.27b-ENVELOPES OF FAILURE IN UNDRAINED TESTS FOR SPECIMENS TESTED IMMEDIATELY AFTER COMPACTION WITH SLOW RATE OF LOADING (14 Days to failure)

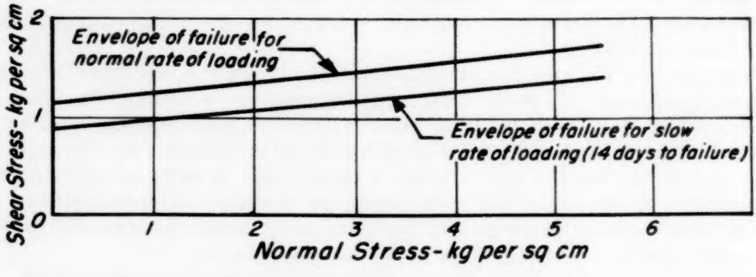


Fig.27c- EFFECT OF RATE OF LOADING ON ENVELOPE OF FAILURE FOR UNDRAINED TESTS ON SILTY CLAY.

compaction to a high degree of saturation at water contents above optimum is desirable or inevitable. It would appear that for some of these soils a thixotropic strength increase with time may have significant effects on their behaviour under load and that consideration of these effects might well be included in design.

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LATERITE SOILS AND THEIR ENGINEERING CHARACTERISTICS

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ABSTRACT

Laterite soils are tropical deposits occurring generally as residual surface deposits. This paper gives a review of existing literature concerning the origin, distribution and physical properties of these soils and in addition includes an evaluation from a civil engineering point of view. The information presented is of considerable importance for the preliminary planning and design of engineering structures in laterite soil regions.

SYNOPSIS

Laterite soils (defined here to include laterites and lateritic soils) are among the most commonly-found tropical soils occurring generally as residual surface deposits. Laterites are generally indurated concretionary deposits formed in situ by the weathering of igneous or sedimentary rocks in tropical countries. It is believed that an essential feature of the weathering process is leaching of one mineral (Silica - SiO_2) and deposition of another (sesquioxides - Fe_2O_3 and Al_2O_3) as a sort of crust. Such a weathering process is referred to as "laterization" and the soils which are laterized to lesser degrees are called lateritic soils.

In this paper, an attempt is made to present an up-to-date review of physical properties as reported by the soil scientist, the geologist and the civil engineer. In addition, this paper gives an evaluation of several properties of laterite soils that are considered important from a civil engineering point of view.

As a result of this study, it is felt that there is a great need for standardizing terminology and collecting more data on physical and engineering properties of laterite soils occurring under different conditions. Such data can be of considerable value in the preliminary planning and design of structures in most tropical countries where construction activity is increasing rapidly.

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INTRODUCTION

The study of soils has been an important subject for the agronomist and the geologist for a long time. During the last century, the knowledge concerning the origin, distribution and nature of soils was an unrelated mass to which mainly the soil scientist, the geologist or the chemist made contributions from their own points of view, and the part played by the engineer was insignificant. Nearly all of the earlier studies, such as those of Dakachaeve(15)* and Glinka,(20) relate to areas which are mostly in the temperate zone, and it was not until much later that similar investigations were made concerning the soils of the tropical zone. One of the most important groups of tropical soils are the residual surface deposits which may be termed as laterite soils. These are found the world over; but principally in the East Indies, southern parts of India, central parts of Africa, northern half of South America and to a lesser degree in many other countries, as shown in Figure 1. Although this map is based mainly on the pedological** classification of soils, which varies somewhat according to different authors,(55,56) it is fairly representative, also from the engineering standpoint, of the occurrence of laterite soils in different parts of the world.

To date there has been no general agreement in literature regarding nomenclature and definitions for the terms relating to laterite soils. In this study, the term "laterite soils" is used in a broad sense for certain surface deposits of the tropics and includes laterites and lateritic soils. The term "laterite" was first used by Buchanan(6) for the brick-red soils found in the southern parts of India. Subsequently it became a practice to refer to all red-colored soils of the tropics as laterites, a practice which has proved to be misleading. In general, laterite soils occur in a variety of forms which range from friable soils on the one hand to almost hard rock on the other—just as limestone occurs in nature in such extreme forms as marl or marble.

Laterites

Laterites are considered to be residual soils of the tropics resulting from the weathering of variety of rocks, such as basalt, gneiss, schist, shak sandstone, limestone, etc., in hot and rainy climates and may be found as indurated strata of varying thicknesses. True or mature deposits often occur in crust-like formations starting somewhat below the ground surface and may continue to a depth of over 100 feet. In the younger and less developed soils the material is relatively soft or friable and has sometimes been called "lateritite." According to Christophe(13) the term laterite refers to a rock-like tropical material red to dark maroon in color and of varied physical appearance. Laterites are known to occur in various forms in different tropical regions. Laterite of India(11) has been described as a sort of red-colored rock with numerous cavities, the cavities being often filled with yellow and red powdery materials whereas laterites of Puerto Rico and Cuba(4,53) are generally dark red, well-granulated materials. Of course, in any large area more than one type of laterite may be encountered.

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*Numbers in parentheses indicate references listed in the bibliography.

**Pedology is the science pertaining to the factors and the laws governing the formation and distribution of soils in nature. Soil science is a much broader subject embracing pedology, soil physics, soil chemistry, etc.

Lateritic Soils

Lateritic soils often occur alongside laterites in areas adjoining the tropics and the subtropics. Unlike laterites, the crust of slag-like concretions or nodules is generally absent. They are plastic to friable soils of red (or even reddish-yellow or yellowish-brown) color. Since these soils are usually very sandy, they are sometimes confused with other red-colored soils which may be of entirely different origin. It is generally possible to differentiate between lateritic and non-lateritic soils on the basis of physical properties combined with a knowledge of chemical composition. Lateritic soils also occur in a wide variety of forms and have been identified by local names such as terra rossa, red loams, "regurs" of India, "tirs" of Morocco, black soils of Australia, etc.

At the present time, due to lack of a sufficient number of engineering studies of laterite soils occurring under different environments, the identification and classification of laterite soils is a difficult task. However, some of the more systematic and properly conducted studies, made for different purposes, often provide valuable information to the engineer. The use, with considerable success, of agricultural, pedological and geological information for the investigation of subgrade conditions for highway and airports has been amply demonstrated.(25) In the following and existing chemical, pedological and geological information concerning laterite soils has first been summarized and then from a study of several of the investigations, most of the important engineering characteristics have been discussed. A knowledge of the expected engineering characteristics of such soils is of considerable importance in conducting soil investigations economically for various civil engineering projects. This is particularly true in most tropical countries where, due to very limited funds, economy is of prime importance.

Review of Literature

a. Soil Scientists' Viewpoints

The soil scientist (and the chemists) perhaps were among the first to study laterite soils and by now numerous studies exist in literature. There is no definite agreement among the soil scientists on a standard chemical composition for laterite soils. However, from a study of the work reported so far, it seems that the presence, in the colloidal fraction, of silica and oxides (usually sesquioxides, F_2O_3 and Al_2O_3) of iron and aluminum, hydrated to varying degrees, is generally admitted. Martin and Doyne(38) believe that the relative proportion of silica to aluminum oxide (as shown in Table 1 below) is more critical than the presence of iron oxide. On the other hand, Pendelton and Shaurusuvana(45) believe that any chemical basis is an erroneous concept and state, as a result of their studies on Siamese soils, that laterite "is a more or less indurated illuvial* quarriable horizon in the soil resulting from accumulation of ferric oxides."

*The terms(26) illuvial and eluvial refer to the movement of soil material from one place to another within the soil in solution or suspension. Horizons that have gained material through such movement are referred to as illuvial and those that have lost as eluvial. The terms generally refer to movements of soil in colloidal form.

It was recognized early by soil scientists like Fermor, Lacroix and Marbut(16,36,37) that there should be some line of demarcation between laterites (true or mature laterites) and lateritic soils. Therefore, silica-alumina ratio (SiO_2/Al_2O_3) in the colloidal fraction, as originally suggested by Harrassowitz(23) and Martin and Doyne,(38) is generally used as basis for separation. Based on this criterion laterite soils are classified as follows:

TABLE 1

	SiO_2/Al_2O_3
Laterite	1.33 or less
Lateritic soil	1.33 to 2.00
Non-lateritic soil	2.00 and over

Although it is recognized that iron oxide is an integral part of most laterites, its inclusions in the ratio with respect to silica does not give so clear cut a constant for identification as does the SiO_2/Al_2O_3 ratio, as shown by Byers and Anderson,(8) Holmes and Eddington(8) and Mattson.(39)

Mohr(43) shows light on the process of laterization by contrasting the leaching effect of rain under tropical and temperate conditions. Under temperate conditions excessive leaching leads to "podzolization," i.e., iron and alumina are leached out (from aluminum silicate) and silica is left behind in the upper horizons. In hot and moist conditions silica is removed and alumina and iron oxide accumulate. Mohr's hypothesis attributes the difference in action to the presence of humic acid. Humic acid, which is abundant under temperate conditions, leaches sesquioxides; while under hot conditions humic acid is oxidized as rapidly as it is formed. In the presence of constant rainfall, temperature conditions alone determine whether lateritic or podzolic weathering would take place. Under temperate conditions, the accumulation of laterite in situ to any great depth is impossible, for if formed at all, it would rapidly suffer detrition and erosion and redeposited as alluvial detritus.(21,52)

Although it is generally agreed that high temperature and moisture conditions of the tropics and subtropics are, in a large measure, responsible for the process of laterization, the exact nature of the mechanism has not yet been very well established.

b. Pedological Studies

So far there is not enough information to separate morphologically the specific features in deposits of laterite soils which could serve as criteria for soil profile determination.(31,40) Many investigators(4,33) have not been able to distinguish between the solum and the parent rock and have extended the solum to hundreds of inches below the surface of the bedrock and have paid scant attention to the surface horizon. However, Harrasowitz(23) reconstructed the following profile from descriptions of Fox and Lacrois(18,36) concerning the laterite deposits of Southern India:

TABLE 2

Orange-red or yellow loam	= about 12 inches
Iron crust with alumina gels (Zone of mottling: with evidence of iron enrichment)	= 100 to 180 inches
Leached zone (rock suffering chemical change, but retaining physical appearance)	= 175 to 300 inches

The red color of laterite soils, though an important physical characteristic, has been the cause of much confusion and hence great care should be exercised in identifying laterite soils on the basis of color alone. Red color is generally more pronounced in laterites than in lateritic soils. Lateritic soils are commonly of lighter shades of red color and, in many areas adjoining the tropics, yellow color is often interspersed. According to Joffee,[34] in the more humid sub-tropic areas one is dealing with superimposition of the process of podzolization on the process of laterization and vice versa. It is very probable, he says, that one is dealing with laterites of an earlier tropical climate which serve now as parent material. We find that red soils, such as terra rosa (red earths) and red loams merging into podzolic and other forest soils in certain parts of the world. For example, the red soils in Greece and Bulgaria adjoin the Chestnut and Brown soils; in France and Italy they are found merging into podzolic and other forest soils; in Palestine area they adjoin the Brown and semi-desert soils and in the United States they are contiguous to Brown Forest soils and the Southern Prairie soils.

c. Geological Investigations

Geologists[7,9,16] have long been interested in laterite soils and have made many studies relating to their chemical composition, especially that of laterites. According to Wadia,[51] laterite is a surface formation composed essentially of a mixture of hydrated oxides of alumina and iron with a small percentage of manganese and titanium. The percentage of the oxides varies widely and hence there are numerous varieties of laterites. At one end of the scale is bauxite with its high percentage of alumina while at the other end are the deposits rich in iron oxides. The predominance of iron oxide in laterites is generally responsible for their red color. The greater proportion of laterites are too ferruginous to be of any use as ores of aluminum and such material has indeed been utilized in the past as iron ore. From a geological point of view, bauxites are merely aluminous laterites. Thus, in India most of the laterites originate from basalt and all the better known bauxite occurrences of India are, according to Brown,[5] are connected with the decomposition of basaltic flows of the Deccan (southern India).

In his investigations, Wadia found that there was very little clay in typical laterites. Usually between the laterite cap and the underlying rock over which it rests there is a lithomarge—a sort of transitional product like rock or bole showing gradual decomposition of the underlying rock.

Engineering Properties

Great difficulty is experienced in determining from the existing literature the various properties of laterite soils due to the fact that different workers have used the same terms for materials of widely different physical characteristics. A complete description is often lacking and hence correct identification is usually difficult. Thus an indication of only the red color is not sufficient. A description of other physical characteristics such as texture, structure, etc., is very important for the identification of these soils. In the following, a few of the physical properties of laterite soils which are of prime importance to the engineer are discussed.

a. Grain Size

Due to the wide range of occurrence of laterite soils their grain sizes, in their natural states, vary a great deal. Laterites, being hard brick-like materials, are usually large aggregates or in nodule-like form while lateritic soils are relatively finer grained. A recent investigation concerning the laterites of India(11) reports the presence of small amounts of silt in the cavities of laterites, in addition to a high percentage of sand.

Studies on lateritic soils by Andrews,(1) Fruhauf,(19) Winterkorn and Chandershekharan(53) and Remillon(48) indicate a heterogeneous grain size distribution which may be approximated as follows:

TABLE 3

	Grain Size	Percentage
Sand	2.0-0.05 mm	50 $\pm$
Silt	0.05-0.002 mm	30-40
Clay	0.002 mm	20-30

In general, the grain size distribution seems to be quite erratic, varying over a wide range very much like glacial till or residual soils. The uniformity coefficient (D60/D20) is expected to be fairly high.

b. Atterberg Limits

Very limited data has been reported for the Atterberg limits of these soils. The results of these tests on a few samples of laterites and lateritic soils from India,(11) French Equatorial Africa(48) and Hawaii,(1) and the range of values are summarized in Tables 4 and 5 below:

TABLE 4

Laterites (Ref. (11) & (48))

	Range
Liquid Limit	30-50
Plastic Limit	25-35
Plasticity Index	5-25

Only the finer fraction from laterites (such as the powdery material from the cavities) was used for such determinations.

TABLE 5

Lateritic soils (Ref. (11), (17) & (48))

	Range
Liquid limit	40-70
Plastic limit	25-50
Plasticity index	15-20

Andrew's(1) data on several samples of Hawaiian laterite soils is not included here because of the results for the Atterberg limits were not obtained by standard test procedures. The Atterberg limits reported by Florenten et al(17) are slightly higher than the range of values shown above. They have also made observation concerning the effect of drying on the Atterberg limits of such soils.

c. Specific Gravity and Density

The results of specific gravity determination reported so far show that the specific gravity of the solids in laterite soils falls commonly in the range 2.70 to 3.50. This is generally higher than for most non-laterite soils, the specific gravity of which lies usually in the range 2.65 to 2.75. The higher specific gravity of solids in laterite soils may be ascribed generally to their higher iron content and, to some degree, to the accumulation of titaniferrous minerals. Some of the determinations made in this connection are summarized below:

TABLE 6

Type of Soil	Specific Gravity, Range	Reference
1. Hawaiian laterite soils	2.73-3.12	Andrews (1)
2. Hawaiian laterite soils	2.84-3.10	Fruhauf (19)
3. Indian laterites	3.00-3.50 (for different fractions)	Central Road Research Institute (11)
4. African laterites (from French Guinea)	2.76-2.89	Florentin et al (17)
5. African laterites (from French Equatorial Africa)	2.79 (average)	Remillon (48)
6. Cuban laterites	2.90	Winterkorn, et al (53)

Very few density determinations (Proctor densities or densities in the natural state) have been made. Two recent determinations given below may give some idea of the expected Proctor densities. In general, relatively high compacted densities could be expected due to high specific gravity of solids.

TABLE 7

Soil	Standard Proctor Max.Dry Density Lbs./cu.ft.	Optimum Moisture Content	Reference
1. Lateritic soil (Guinea, Africa)	113	10.3	Winterkorn, et al (53)
2. Laterite (Morroco, Africa)	126	13.9	Remillon (48)

In a recent paper Florentin et al[17] report natural dry density of several samples of laterites originating from granite, dolerite and schist. The average dry densities (in pounds per cubic foot) were found to be 79, 80 and 93.

d. Permeability

Of the wide variety of physical forms in which laterite soils are found, the porous and granular types are naturally the most permeable. In general, the permeability of laterites is on order of 10^{-2} to 10^{-1} cm per second, depending on their physical aggregation. Hence, laterites usually have the drainage properties desired for pavement subgrades. Similarly the granular varieties of lateritic soils have a high drainability in situ but become clayey and plastic to the depth of disturbance when worked (or remolded) in the presence of water, and hence exhibit low permeabilities.

e. Presence of Minerals

Laterite soils contain most of the common minerals, such as silica, alumina, oxides of iron, etc., which are usually found also in non-laterite soils.[22,40,41] The presence of clay minerals, such as kaolinite, illite, montmorillonite, halloysite, etc., has been reported in laterite soils in very small quantities. Montmorillonite usually occurs in traces only. Thus due to lack of clay minerals in sufficient proportions, laterite soils are generally not active.

f. Swelling Properties

Laterite soils commonly swell only slightly, if at all. It has not yet been possible to establish clearly any criteria on the basis of which one could predict precisely the swelling properties of these soils. The presence of clay minerals such as illite and montmorillonite which is known to influence the swelling properties of many other soils[30,41] seem to affect the behavior of these soils in a like manner. As indicated above, since such clay minerals are usually absent, laterite soils, especially laterites, do not generally exhibit any appreciable swelling in the presence of water. However, soils possessing low degrees of laterization are found to possess more pronounced swell and shrinkage characteristics. Thus some clay-like lateritic soils, such as the black soils of India, called "regur," or Morocco (locally known as "tirs"), and of Australia, are known to have relatively high swelling and shrinkage characteristics.[29]

Engineering Behavior

Laterite soils are expected to play an important part in the expanding construction activity in several countries of Asia, Africa and South America. It is therefore essential that the engineers associated with such activity have some idea not only of the physical properties but also of the engineering behavior of these soils in the field. Two principal engineering uses of these soils, viz., subgrade for pavements and foundation for structures, will be discussed here. The use of these soils as materials of construction will also be indicated briefly. At present there is not enough data to warrant a correlation between laboratory tests and a field performance of such soils, but with further investigations, a satisfactory correlation for any given type of laterite soil should be quite possible.

Sometimes one comes across a mistaken notion that all laterite soils are troublesome materials and should be avoided. As is apparent from this study, such a generalization is not correct and reflects a lack of understanding of the different characteristics of laterite soils. In general, the suitability of any type of laterite soil deposit can usually be judged by a systematic study of different engineering characteristics of the soil, relevant to a given structure. One should be cautious in applying the routine laboratory or field soil tests to laterite soils. The application of such tests should be preceded by a careful study of their properties, for, due to some of their unusual characteristics, commonly-used soil tests may have to be modified considerably.[12,19]

a. Subgrade and Surfacing for Pavements

Laterites in their natural state generally act as good subgrades for pavements carrying moderate traffic loads. Laterites are used extensively for base and sub-base course for secondary roads and other low cost pavements in southern India. The base course is generally prepared as a simple water-bound macadam by the use of a light roller which results in a smooth, hardened surface. Often no surfacing is laid over the base course, especially if very light traffic is expected. However, since laterites do not possess sufficient resistance to abrasion, their use as a base course material under heavy traffic loads is not recommended. Fruhauf in his studies on laterite soils of Hawaii found that such soils in natural undisturbed state are very friable and, if carefully handled, can produce a good highway subgrade of ample bearing capacity. However, due to the pulverizing action and consequent remolding in the presence of moisture by heavy construction equipment, these soils are transformed into a highly plastic material which generally has a low bearing capacity and poor drainage.

In recent years reports from Indian,[42] Equatorial Africa[13,14,35] and Paraguay[10] indicate a successful use of laterites and asphalt cut-backs for low cost surfacing. The surface dressing is done by one of the several methods using a suitable cut-back. Usually a suitable grade of asphalt (such as MC-2) is applied by penetration method over a laterite macadam.

b. Foundations of Structures

The behavior of laterite soils as foundations of structures is as varied as their occurrences. In laterite deposits, it may be possible to build ordinary structures on suitably designed footings located a few feet below the ground surface; however, heavier structures may have to be based on firm rock. One

must ensure the soundness of the rock by suitable exploratory methods as the upper part of the rock below laterite is likely to be decomposed. Due to induration of the deposits pile driving is generally a very difficult operation in laterite deposits and one can rarely drive piles more than a few feet. In cases where loads of structures have to be taken to a deeper and sound stratum, pre-excavated piles or a caisson foundation may have to be considered.

Lateritic soils are generally expected to behave like most fine-grained soils. If the lateritic soil in question is more clay-like, then its bearing capacity can usually be determined by unconfined compression tests performed in the laboratory on soil samples from different depths or a suitable field test such as the standard penetration test* which can give a sufficient indication of the stiffness or relative density of the soil at different depths. However, if such a soil deposit is predominantly silty, great care is needed to determine its shear strength by triaxial tests performed in the laboratory on several undisturbed samples.

c. Materials of Construction

Depending on the type of laterite soil and its use, laterite soils can be poor to excellent materials of construction. There is ample evidence to disprove a theory that all laterite soils are poor materials for construction. In fact, in the past some of the harder varities of laterite soils have been used for building and road construction. In Thailand, several temples(45) built from masonry blocks of laterites are still standing after experiencing ravages of a tropical climate for over a hundred years. In India and Burma(53) laterite is used as road metal or base material for highways with moderate traffic loads. For the construction of dams Van Es(50) and Florentin et al(17) have utilized, after due study, forms of laterite soil available locally.

In more recent times, a few studies have been made concerning the stabilization of different types of laterite soils for highway construction. It has been found, by studying laterites from French Equatorial Africa,(48) that the harder laterites (free from loam or clay) have a good affinity for asphalts and cut-backs. In general, stabilization of laterite soils using common admixtures (such as portland cement, cut-back asphalts, etc.) does not seem to present any special difficulties. From his laboratory studies on different laterite soils, Winterkorn(53) has shown that their susceptibility to stabilization may vary from poor to excellent. In general, this susceptibility increases with the degree of laterization as evidenced by the silica-sesquioxide ratio.

CONCLUDING REMARKS

In the foregoing, the origin, nature, distribution, and physical properties of laterite soils have been presented. The investigations analyzed here are mostly the work of the soil scientists and the geologists but includes a few

*The sounding test method in which the number of blows of a 140-lb. weight falling vertically through 30 inches are noted for one foot penetration of the standard split spoon. The standard split spoon has inside diameter of 1-3/8" and outside diameter of 2" and is usually 2 to 2-1/2 feet long.

which have been reported from an engineering point of view. There is a dearth of sufficient data on the physical properties of various types of laterite soils formed under different environments. Hence, the need for studies which can give rise to data of use to the engineer cannot be over-emphasized. If enough data on the physical properties and field performance of these soils is accumulated, it may promote correlation of some of these properties with anticipated engineering behavior.

The information presented in the preceding is a brief introduction to an understanding of laterite soils but it may be useful in the preliminary planning and design of engineering structures such as highways, dams, airports, etc., in countries involving the use of laterite soils. This information does not obviate the necessity of making a detailed soil exploration at the particular site selected for an engineering structure. However, it does give a general idea as to what might be expected of such soils, thereby reducing considerably, in most cases, the scope of a soil exploration program. The absence of such information, one not familiar with such soil deposits, may spend considerable time and effort to ascertain their engineering characteristics.

One great difficulty experienced in studying the available literature was the lack of uniformity in nomenclature. Different authors have assigned a variety of definitions and meanings to the same term, fiz., laterite. It would be profitable if investigators from different fields would have a joint conference to standardize definitions of terms relating to laterite soils. One of the best approaches would be the one based on examination of the complete profile[47] and the determination of the hydrated alumina in the soil, i.e., an effort to correlate the physical and chemical properties.

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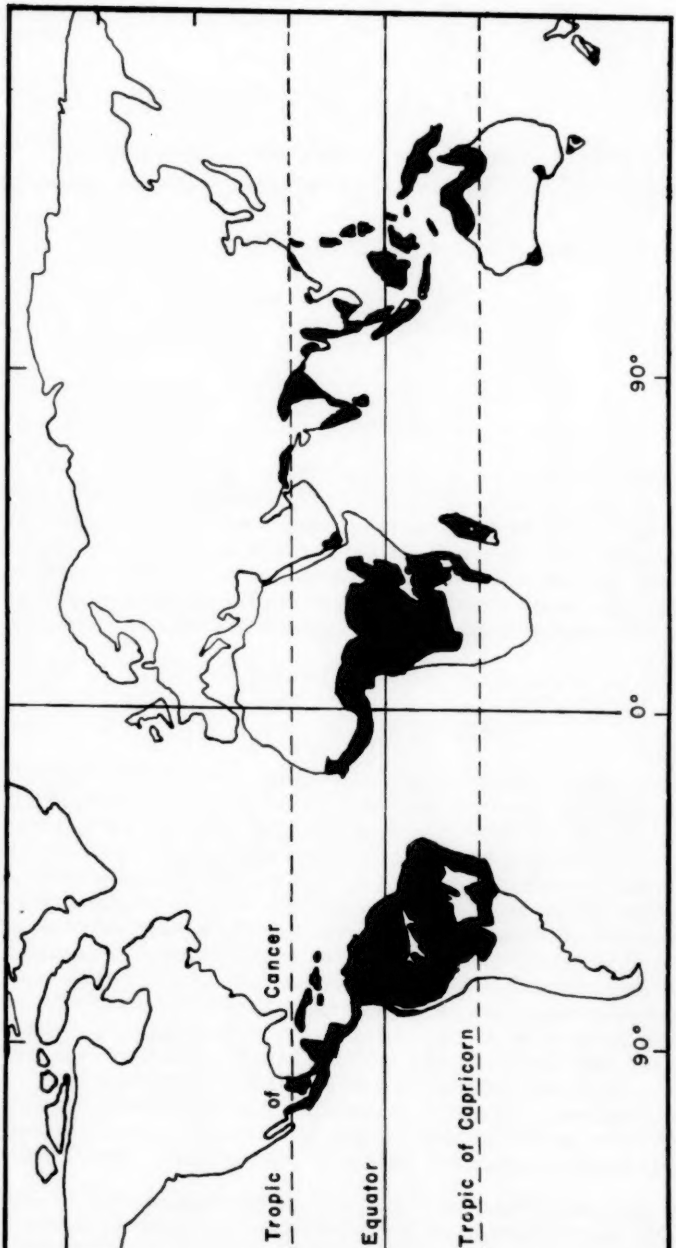

WORLD MAP
SHOWING DISTRIBUTION OF LATERITE SOILS (DARK AREAS)
From Goode's World Atlas (Rand McNally & Co., 1955)
FIGURE-I

Paper 1429 November, 1957 SM 4

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GEOLOGIC INVESTIGATIONS OF DAM SITES BY THE SCS

Gunnar M. Brune,[1] Aff. M. ASCE
(Proc. Paper 1429)

ABSTRACT

"Geologic Investigations of Dam Sites by the SCS," by Gunnar M. Brune. (SM) The role of geologists in the Soil Conservation Service, particularly in connection with its dam building program, is discussed. Types of drilling equipment used, organization of parties, and costs of drilling are dealt with. Drilling investigations are divided into four major phases: foundation, emergency spillway, borrow material, and embankment drainage and special investigations. Geologic reports and laboratory analyses are also discussed.

INTRODUCTION

With acceleration of watershed projects, starting in November 1953, geology was given an important part in the planning and design of engineering works. Two branches of geology have been established in the Soil Conservation Service: watershed geology and engineering geology.

Watershed geologists in the Soil Conservation Service are concerned primarily with the erosion, entrainment, transportation, and deposition of sediment. They make studies of sources of sediment, sediment yields from watersheds, deposition in reservoirs and channels and on flood plains, sediment routing and delivery rates, aggradation and degradation of channels, and the effect of the Service's program upon sedimentation problems. These types of geologic studies, however, are outside the scope of this paper. They have been very well covered previously by Gottschalk and Jones.(7)

The engineering geologist in the Soil Conservation Service, is engaged in geologic studies related to engineering design and construction and ground water problems. He makes geologic investigations for stabilizing structures, dikes, levees, tidegates, and irrigation, drainage, and other channels. He

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1. Eng. Geologist, Eng. and Watershed Planning Unit, Soil Conservation Service, Fort Worth, Tex.

advises on the geologic aspects related to the construction and repair of farm ponds, particularly in regard to methods of preventing leakage. He works closely with the irrigation engineer in the development and recharge of ground water reservoirs. He assists in the development of seeps and springs for farm and ranch water supply. Finally, and this is the subject of this paper, he makes geologic investigations for various types of dams built by the Soil Conservation Service.

As Burwell and Roberts(5) have so aptly stated: "Engineering Geology is not a branch of the science of geology; it is the application of all of the branches of the science to the practical problems of engineering. Physiography, historical geology, stratigraphy, structural geology, petrography, economic geology, ground-water hydrology, and even paleontology—all these divisions of the science have important applications in civil engineering. The basic difference in requirements of the geologist who is to serve the civil engineering organization from the investigator or teacher in pure geology is one of knowing more about the fundamentals and practical requirements of engineering and of being temperamentally equipped to deal with the practical problems of engineering rather than of knowing more about certain branches of the science of geology."

This applies as much to engineering geologists in the Soil Conservation Service as it does to those employed by other engineering organizations. The dam building activities of the Soil Conservation Service have developed rapidly in recent years. Under various programs the Soil Conservation Service provides technical assistance to local cooperators in building many types of dams, including farm ponds, debris basins, desilting basins, irrigation reservoirs, floodwater retarding structures, and multiple-purpose dams which may include municipal or industrial water supply.

These structures range considerably in size. As an example, one of the structures is a multiple-purpose dam under construction six miles southeast of Marlow, Oklahoma. This is a combined floodwater retarding structure and water supply for the city of Duncan. A few statistics on this reservoir follow:

Drainage area	32.17 square miles
Storage capacity:	
Sediment	1.96 inches, or 3,320 acre-feet
Municipal	6.22 inches, or 10,650 acre-feet
Floodwater	9.48 inches, or 16,230 acre-feet
Total	17.66 inches, or 30,300 acre-feet
Height of dam	65 feet
Volume of embankment	682,500 cubic yards

Those interested in a complete history of flood prevention and watershed protection in the Soil Conservation Service, and a detailed discussion of the types of structures being built, are referred to Matson's paper on Construction and Agriculture.(10) The writer(3) has also described previously some of these structures built in western Iowa.

Organization and Equipment Used

The Soil Conservation Service recognizes that adequate site investigation is a necessary adjunct to design and construction. Many of the states working with the Soil Conservation Service are equipped with powered drilling

machines of various types. These range from small portable chain-saw power augers to core drill rigs capable of drilling to a depth of 1,500 feet.

Core drills are equipped with barrels and bits up to 7-3/4 inches in diameter, with which 6-inch rock cores and undisturbed "Denison" samples are taken. Denison samples are obtained in cylindrical sheet steel liners. Wooden plugs are placed in both ends and the ends sealed with paraffin to prevent moisture loss during shipment to the laboratory. Diamond bits of various types are used in the harder rock formations and "crackerjack" bits in medium hard formations.

Push-tube equipment has been installed on some rigs, but has not proved very satisfactory, because the samples obtained are often considerably disturbed. Drive samples are disturbed even more, and therefore are not much used. Drilling with compressed air is usually not practical, as it is successful only in dry materials. Water usually is encountered at some depth in dam-site investigations. Spiral augers have been replaced largely by enclosed, bucket-type augers in order to reduce to a minimum the mixing of material from the hole.

Trenching with a dozer, shovel, or backhoe is sometimes used in exploratory work, in formations which are difficult to drill, such as gravel and cobbles, and to give contractors a better picture of excavation problems than they could obtain from inspection of cores.

In states in which the volume of construction is small, drilling investigations are done under contract by drilling companies. In other cases drilling parties may serve more than one state. With the constantly increasing volume of dam construction, more and more Soil Conservation Service state offices are purchasing drilling equipment and organizing their own drilling parties.

The larger drilling rigs are equipped so that they may be used for installation of relief wells in dams. For this purpose it is necessary to use 18-inch casing and to drill 20-inch holes with auger or fishtail bits.

Drilling parties differ in size. The larger parties such as those located in Texas and Oklahoma are most efficient and economical for the volume of work and size of structures installed in those states. These parties usually are equipped with a large rotary core drilling rig, a water and tool truck, a power-driven soil auger, a carryall, and two passenger cars. Personnel consist of two geologists, a driller, a junior driller, and three laborers.

The annual cost of running one of these large drilling parties may be summarized as follows:

Salaries (7 men)	$23,420
Per Diem	15,120
Fuel and Lubricants	2,480
Depreciation (12-1/2 percent)	6,100
Maintenance	2,930
Supplies	1,800
Total	$51,850

A party completes about one dam site investigation per week, or 48 sites a year, allowing for leave and travel time. Thus, the cost per site for geologic investigation averages $1,080. To this must be added about $30 for clearing to make the area accessible to the drilling equipment, and $237 (average of 105 dam sites) for laboratory analysis of samples. Hence the total cost per site averages $1,347.

The average footage drilled per site is 523 feet. Thus the drilling cost is $2.07 per foot.

Geologic investigations for floodwater retarding and other dams usually fall under four major headings: Foundation, Emergency Spillway, Borrow Material, and Embankment Drainage and Special Investigations. Detailed discussions of Soil Conservation Service methods of investigations for specific areas have been prepared previously by Griswold[8] and Brune.[4] The following discussion sets forth in general the procedures and techniques used in the Soil Conservation Service. All geologic investigations are made with the close cooperation of the field engineers.

Foundation

Usually the drilling party starts by drilling holes on 100- to 200-foot centers along the centerline of the dam and along the centerlines of any outlet structures. Additional holes are drilled if necessary to establish the continuity of the strata. Figure 1 shows a typical distribution of holes along the centerline of a floodwater retarding dam (now completed) near Bruceville, Texas.

Continuous Denison (undisturbed) samples are taken from at least one hole. These are shipped to the Soil Mechanics Laboratory in Lincoln, Nebraska, for such tests as consolidation-settlement, shear, and permeability. Large (25-pound) disturbed samples may also be taken. These are used for compaction and other tests, to determine usability of material taken from the core trench in the embankment. In addition to the auger bit and Denison barrel, a rock core barrel is used where necessary to determine depth of weathering of rock and best core trench depth. "Split" Denison samples are taken in order to log unconsolidated material more accurately than when it is disturbed by augering. In this case the Denison tins are opened for visual inspection by the geologist rather than being sent to the laboratory.

Because wet excavation or dewatering a foundation area with a system of well points can be very costly, dams are often designed so as to avoid these items. The geologist advises as to the practical core trench depth. He also recommends drainage measures to take care of seepage.

The foundation materials available are delineated. The geologist keeps in mind that larger concrete structures usually require firm rock foundations, whereas the lighter concrete structures and earth embankment may often be placed directly on softer materials such as clay or shale.

Because blasting is usually not advisable in the core trenches or keyways, the geologist is careful to recommend core trench and keyway depths such that no firm, unweathered rock must be removed.

Materials which may suffer excessive foundation settlement, such as cavernous gypsum, are always explored in detail. Cores are taken of shales, especially the compaction type, for slaking (wetting-drying) and freezing-thawing tests. Foundations of these materials may require special treatment such as spraying with asphalt, or immediate fackfilling of the core trench upon exposure. Rebound following unloading is also a serious problem in some types of shale.

The geologist looks for bedding planes separated by layers of clay or soft shale, especially those inclined downstream, as they may, under certain conditions, serve as sliding planes and result in failure.

Potential slides and shear zones are always explored and delineated in detail. Weak bedding planes or joints may destroy much of the inherent shear

strength of a rock and reduce the problem to one of sliding friction. This problem has been discussed in detail by Burwell and Moneymaker.(6)

Tuffs and silty materials are among the worst offenders in regard to sliding tendencies. Beds of these materials which may be subject to rapid draw-down or which occur in either abutment are suspect, and call for special drains or other measures in the design of the dam to prevent slides.

Faults, because they afford potential sliding planes and permeability problems, are often sufficient cause for relocating a dam site. However, a study usually is made first of earthquake records and other evidence to determine whether the fault is still active. If it does not appear to be active, and if the fault zone is not shattered the proposed dam may be located so that its concrete structures are not situated over the fault.

Emergency Spillway

In the open spillway investigation, the drilling party generally puts down at least ten holes or dozer trenches. More are required if much rock is encountered above spillway grade. These holes or trenches normally are placed along cross sections to facilitate computation of rock and common excavation. Figure 2 shows a typical system of borings on spillway cross sections for a floodwater retarding dam near Brownwood, Texas. Note that in this case the spillway was relocated in order to reduce the amount of rock excavation.

Large disturbed samples are taken for compaction and other tests to determine whether the material to be excavated is suitable for use in the embankment. Rock cores up to six inches in diameter are taken and usually left at the site for inspection by prospective contractors. Easily weathered cores such as those of compaction shales are coated with paraffin and taken to a nearby Soil Conservation Service office for protection against weathering. If there is doubt as to the usability of rock for riprap or other purposes, cores are sent to the laboratory for wetting-drying (slaking) and freezing-thawing tests.

One of the major purposes of the open spillway investigation is the determination of the volume of rock excavation that will be required. If this is excessive, it may be necessary to relocate the spillway, place it at the opposite end of the dam, divide it between both abutments, or raise the dam and spillway so as to store a larger quantity of runoff and reduce spillway discharge requirements. If any relocation is done, the new spillway must be drilled.

The Soil Conservation Service defines rock excavation (Blackert 1) as "all materials required to be excavated for the permanent structures, within the limiting lines and grades shown on the drawings, or as established by the Engineer, which, in the opinion of the Engineer, cannot be removed by a ripper or rooter unit weighing eight thousand five hundred (8,500) pounds, equipped with a hard-faced steel tooth, as recommended by the manufacturer, and drawn by a track-type tractor of one hundred (100) draw-bar horsepower, in good condition and capable of developing twenty-eight thousand (28,000) pounds draw-bar pull in first gear, and shall also include all boulders and detached rock one (1) cubic yard or greater in volume. In areas where blasting is not permitted, materials of rock character which require the use of power operated drills for removal will be classified as rock excavation."

The geologist is responsible for classifying material to be excavated as rock or common excavation. Also he usually divides the common excavation

into two classes: (1) Unconsolidated soil and rock fragments up to six inches in diameter, which may be mixed with earth fill for the embankment, and (2) rock fragments and boulders from six inches in diameter to one cubic yard in volume, which require special disposition in the design and construction of the dam.

The geologist makes recommendations as to the use of the various materials in the dam, particularly in regard to rock. Rock is often used for riprap. Other uses are for filters, drains, and roadways. In some cases, where soil materials are scarce, dams are built almost entirely of rock, following essentially the design principles laid down by Bleifuss and Hawke.(2) Figure 3 shows an example of the use of rock in a floodwater retarding dam near Charleston, Arkansas.

Tentative recommendations as to the resistance of various rock materials to erosion and weathering are made, subject to more exact determinations in the laboratory. The erodibility of materials underlying the emergency spillway floor must be determined. In the case of soft, erodible materials, the spillway is usually cut six inches to one foot below grade, backfilled with topsoil to grade, and seeded to protective grasses. In the case of resistant rock, this procedure is not necessary.

Since emergency spillways often involve cuts 40 feet or more in depth on the uphill side, consideration must be given to the possibility of slides. Where the dip of the strata is toward the cut, especially in shale and clay beds, a cut slope flatter than the dip is usually recommended. Slides in spillways are apt to partially block the spillway channel.

Borrow Material

Borrow investigations are usually made on a grid system, such as is shown in figure 4 for a floodwater retarding dam near Leedey, Oklahoma. Holes are augered to depths up to 20 feet. Sufficient large (25 pound) disturbed samples are taken to represent all of the types of material encountered.

The geologist makes tentative recommendations as to the use of materials in the embankment, pending the results of detailed laboratory tests.

Some types of borrow material, such as volcanic ash or tuff, montmorillonite clays, dispersed soils, gypseous soils, or those high in silt, are used only with caution. Such materials have a very light volume weight or are subject to excessive swelling, piping, settlement, or slippage. Special measures may be recommended by the geologist to counteract these tendencies, such as mixing with better quality material, flattening the side slopes of the dam, or using heavy rollers for adequate compaction.

Of equal importance in borrow investigations is the study of ground water conditions in the flood plain area. Drill holes are bailed and periodic water level readings taken to determine both the rate of water movement through pervious strata and static ground water level. Information is also gathered as to the seasonal fluctuation in ground water table.

Where the borrow area is underlain by pervious material such as sand, fractured or weathered rock, or cavernous limestone, the permissible depth of borrow is limited so as to leave a blanket of relatively impervious material on top of these beds, thus preventing excessive water losses.

The geologic investigation also includes a study of possible sources of water for use in construction. These may be nearby ponds, rivers, wells or

lakes. If the ground water table is shallow, the contractor may elect to dig a sump in the flood plain to obtain water. Sometimes, as in the case of abandoned oil well water high in salt content, samples may be required for laboratory testing in order to determine whether the water is satisfactory for use in construction.

Embankment Drainage and Special Investigations

In the case of floodwater retarding structures, it is not essential that the dam or reservoir be watertight. However, seepage must be carefully controlled so as not to endanger the dam.

The geologic investigation locates and delineates any pervious formations such as cavernous limestone or gypsum, scoriaceous lava flows, open joints or bedding planes, sand and gravel beds, buried stream channels, or pervious glacial deposits, either under the dam or around the reservoir rim. Often dyes are used to trace the movement of underground water beneath proposed or existing reservoirs, following the methods described by Legget.(9) Dyes such as those used in Air Force survival kits have been found to be satisfactory.

Usually borings are made on several cross sections of the stream channel where it passes under the proposed dam. The depth of loose sand and gravel deposit is determined. If these deposits appear to be suitable for use in foundation drains, filters, or roadways, small (4-pound) disturbed samples are taken for mechanical analysis by the laboratory.

Depending upon the problems encountered, additional exploratory holes are put down. If grouting appears to be required, additional drilling is done to determine the spacing and depth of grout holes and the probable volume of grout needed.

If deep, porous strata extend to great depths beneath a dam, a positive cutoff of seepage water by a core trench may be impracticable. In such a case, relief wells may be advisable. Exploratory borings are made about 25 feet below the downstream toe of the proposed dam. Continuous Denison samples are taken for submission to the soil mechanics laboratory. The depth and thickness of all aquifers through which leakage might occur are determined, and recommendations as to location and depth of relief wells made.

If pervious strata occur at shallower depths, foundation drains of various types may be recommended instead of relief wells, and the method of investigation varied accordingly.

In the case of possible slides in spillway cuts or elsewhere, French or other type drains may be recommended to divert water away from the slide areas. The geologist must do enough exploration to fix the best location and depth of such drains, and also to locate sources of filter pack material if possible.

Reports and Laboratory Tests

The drilling party geologist prepares a report on each dam site investigated, which summarizes location, topography, general geology of the site, drilling equipment used, details on method of investigation, findings, and recommendations on classification of rock excavation or on any special design features or construction measures believed to be necessary in view of the geologic conditions found.

A copy of the report is sent to the design engineer and a copy to the soil mechanics laboratory. The soil mechanics laboratory makes the detailed tests required for the particular site. These may include mechanical analysis, dispersion, percent gypsum and other salts, Atterberg limits, shear, compaction, consolidation-settlement, permeability, volume weight, slaking, freezing-thawing, swelling, percent organic matter, percent calcium carbonate, or others.

The laboratory depends upon the findings of the geologist to indicate what special laboratory tests may be required, but recommendations made by the geologist in the field may be modified or refined by the laboratory on the basis of its detailed tests. The laboratory also prepares a report on each dam site, a copy of which is sent to the design engineer. The design engineer uses both the geologic report and the laboratory report in preparing the final design for each structure.

The Soil Conservation Service has found that geologic investigations and laboratory tests not only are essential to insure the safety and stability of the structures, but by obtaining adequate advance information on site conditions and characteristics it may be possible to effect substantial savings in construction cost. The average cost of site investigations and laboratory tests is 3.0 percent of the total cost of a structure.

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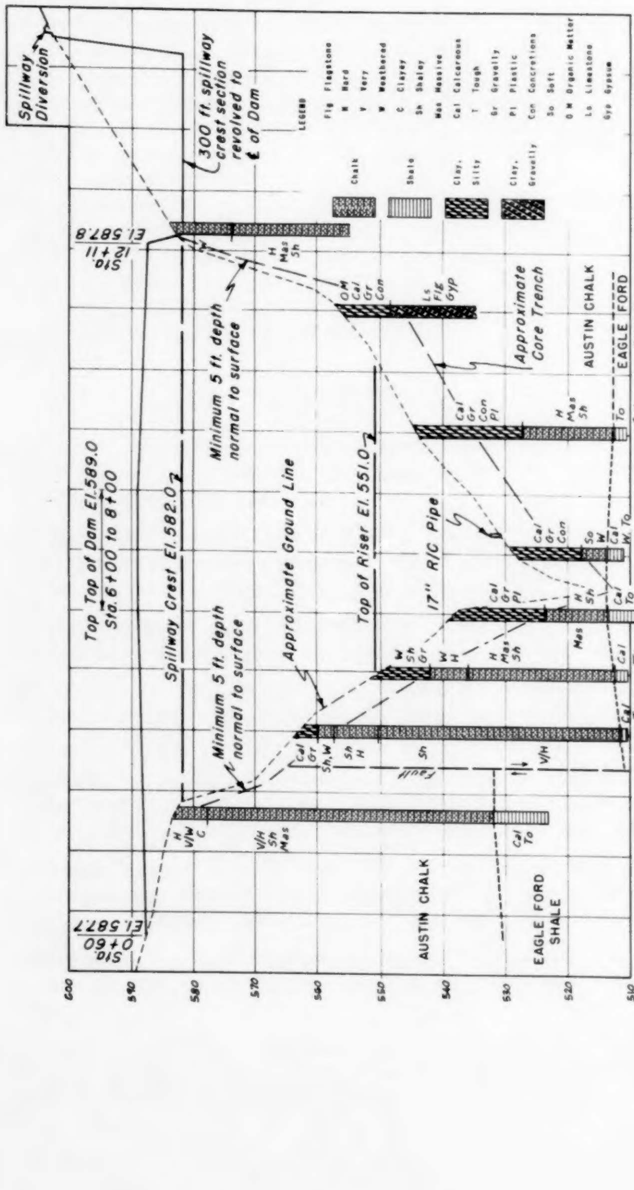

Figure I

TYPICAL PROFILE ON CENTERLINE OF DAM
FLOODWATER RETARDING STRUCTURE SITE NO. 5
COW BAYOU WATERSHED NEAR BRUCEVILLE, TEXAS

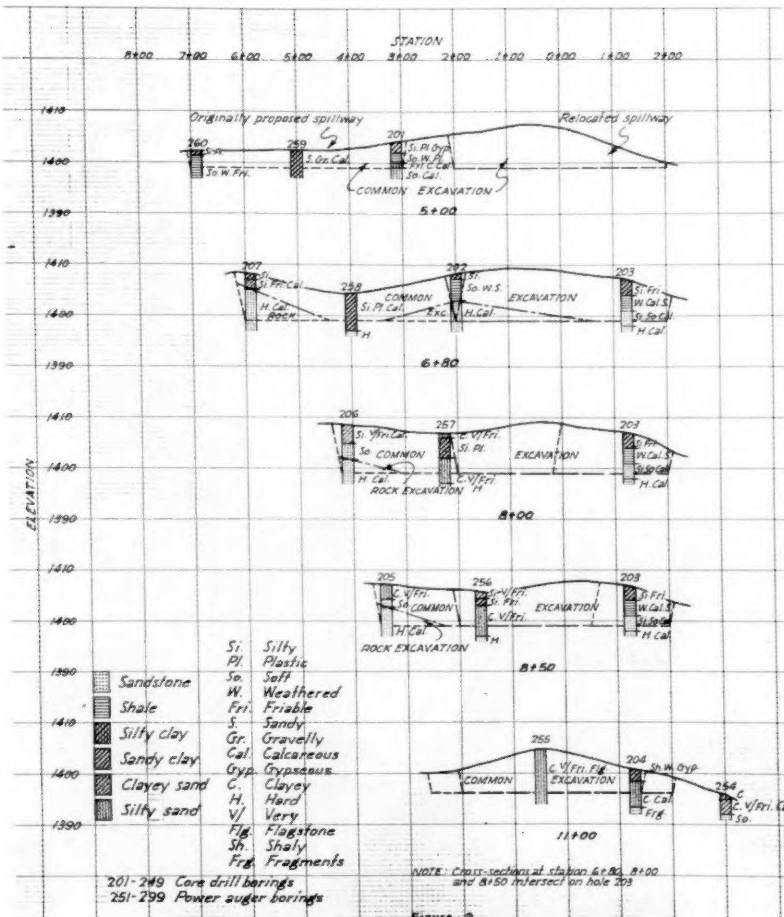

Figure 2

EMERGENCY SPILLWAY CROSS SECTIONS, WITH LOGS OF BORINGS

FLOODWATER RETARDING STRUCTURE SITE NO. 7

CLEAR CREEK WATERSHED NEAR BROWNWOOD, TEXAS

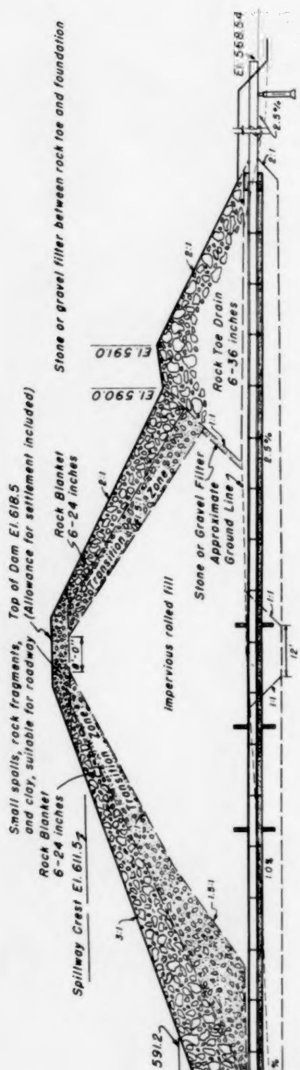

Figure 3

TYPICAL SECTION THROUGH DAM SHOWING USE OF ROCK

FLOODWATER RETARDING STRUCTURE SITE NO. 20

SIX MILE CREEK WATERSHED NEAR CHARLESTON, ARKANSAS

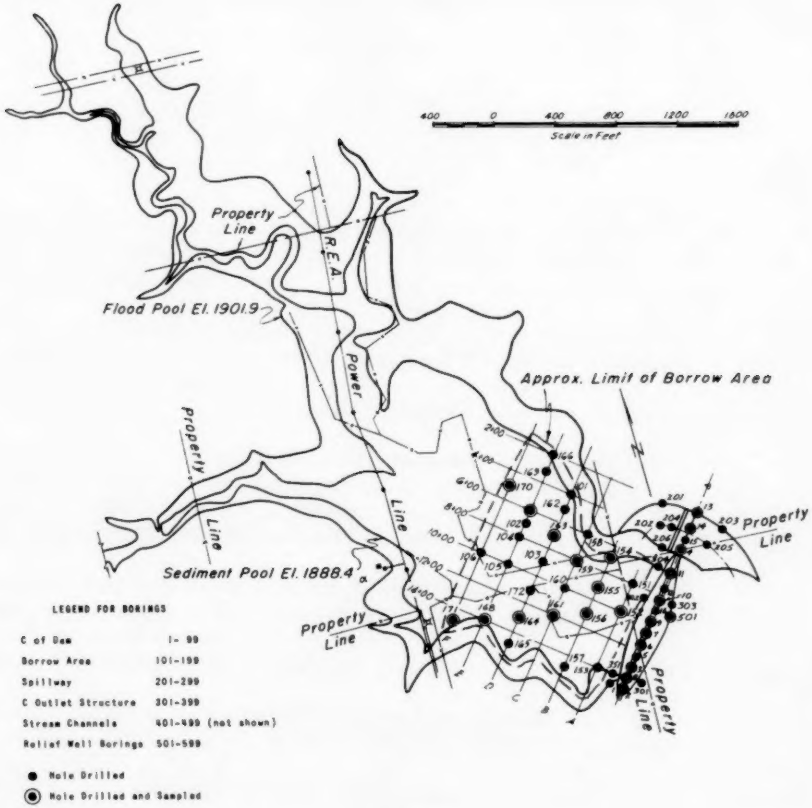

Figure 4

TYPICAL PLAN OF RESERVOIR AND BORINGS
FLOODWATER RETARDING STRUCTURE SITE NO. 1
BARNITZ CREEK WATERSHED NEAR LEEDEY, OKLAHOMA

U.S.D.A. SOIL CONSERVATION SERVICE, FORT WORTH, TEXAS 1-29-57 4-R-11,168

Journal of the

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1430)

(Over)

* There will be no closure.

PILE TESTS, LOW-SILL STRUCTURE, OLD RIVER, LA.[a]

Closure by C. I. Mansur and R. I. Kaufman

C. I. MANSUR,[1] M. ASCE and R. I. KAUFMAN,[2] J.M. ASCE.—The interest in the pile loading tests shown by Messrs. Nishida and Gisienski in their discussions is appreciated.

It is possible that the coefficient of earth pressure K and unit skin friction in silt was greater in the compression tests than in the tension tests as a result of the "swelling" of the pile in compression as proposed by Mr. Nishida. In the tension tests the average value of K in silt was about 0.6, about equal to an at-rest value, while in the compression tests the average value of K in silt was about 1.6, approaching a value of passive earth pressure.

Mr. Nishida has raised the question as to whether the shearing strength of the silty soils is comprised, in part, of cohesion. A shear strength of about $\phi = 28$ degrees, $c = 0.1$ ton/sq. ft. was obtained from consolidated-undrained triaxial compression and consolidated-drained direct shear tests on undisturbed specimens of silt; a shear strength of about $\phi = 33$ degrees, $c = 0$ was obtained from consolidated-drained direct shear tests on undisturbed specimens of silty sand. As the cohesion was relatively small it was neglected in the design of the pile foundation and analyses of the pile loading test data.

Mr. Gisienski has questioned the effect of driving closely-spaced wood anchor piles (on 2.5-ft. centers) on the density of the foundation soils and whether possible differences existed between the density of the foundation in the pile test excavation and that of the foundation for the structure where no anchor piles are to be driven. The timber anchor piles had an embedded length of about 44 ft. and therefore penetrated only into the silty portion of the foundation. Although it is probable that driving these piles caused the silty soils to densify, no measurements were made to determine this effect. Measurements made when driving the piling for the structure indicate that the bottom of the excavation settled about 0.5 to 1.0 ft. as a result of driving the structure piling (14 in. H-beams) which is spaced on about 5-ft. centers. It is believed that this settlement is primarily the result of densification of the silty portion of the foundation and that generally conditions at the test piles in the pile test excavation after the anchor and test piles had been driven would not differ significantly from those at the structure after all structure piling had been driven. Furthermore, as the piles were designed primarily to support the design load by point bearing and skin friction in the deep sand stratum with an adequate factor of safety, differences in density in the upper silt stratum beneath the test excavation as compared to those beneath the structure were considered of minor significance with respect to the load supporting capacity of the portion of the pile embedded in sand.

It is considered that the sequence of driving the test piles did not

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a. Proc. Paper 1079, October, 1956, by C. I. Mansur and R. I. Kaufman.
1. Independent Wellpoint Corp., Baton Rouge, La.
2. Mississippi River Comm., Vicksburg, Miss.

significantly affect their penetration resistance and static capacity, and that variations in these resistances and capacities reflect the variability of in-situ conditions. As noted by Mr. Gisienski the final penetration resistance of pile 6 (19 in. diameter) was about 60% greater than for pile 2 (21 in. diameter). Pile 2 was at the center of the row of test piles comprised of piles 1, 2, and 3, see fig. 3. Pile 2 was driven after piles 1 and 3. Pile 6 was at the end of the row of test piles comprised of piles 4, 5, and 6 and was the first pile driven in this row. Pile 6 was driven several days prior to driving pile 2. Had the sequence of driving had a significant effect on driving resistance, it would be reasonable to expect that pile 2 would have had a greater driving resistance than pile 6. However, final penetration resistance of pile 6 was greater than that of pile 2. Furthermore, pile 4 (17 in. diameter) offered the highest resistance to driving of all piles and had the greatest failure load in sand. This pile was at the end of a row of piles, see fig. 3, and was driven after pile 6 but before pile 5. Pile 7 was the last test pile driven. The anchor piles were driven subsequent to driving the test piles. Thus when pile 4 was being driven, closest pile which had already been driven into sand was 25 ft. away. Based on the above, it is considered that the sequence of pile driving had no noticeable effect on the pile driving resistance.

Mr. Gisienski also has inquired about the procedures used to determine the tip movement of the pile and "residual" stresses in the piles in the tension tests. The tip movement of the pile was determined from the movement of the bottom strain rod anchor with the pile loaded. The net movement of the pile was determined as the residual settlement or rise remaining after the test load was removed from the pile and the pile was permitted to rebound. In general the net movement agreed closely with the tip movement at a given load, although some difference was noted as shown on figs. 10 and 15. The "lock-in" or "residual" stresses and their effect on tension test data are believed to have developed in the soil and pile in the compression tests which were performed prior to the tension tests. It is believed that piles loaded in compression to the ultimate capacity of the foundation developed high compressive stresses and strains which were not relieved completely upon subsequent unloading of the pile. Loading these piles in tension would tend to further relieve the compressive stresses, although based on tension test data for piles 4 and 6 on fig. 14 it appears that sections of these piles in the vicinity of and immediately above the top of sand may still have been in compression during the tension tests. Had the residual compressive stresses been relieved completely in the tension tests, or had the tension tests been performed prior to the compression tests, it is believed that the "load in pile" curves for the tension tests on fig. 14 would have been less erratic.

SOIL MODULUS OF LATERALLY LOADED PILES[a]

Closure by Bramlette McClelland and John A. Focht, Jr.

BRAMLETTE McCLELLAND[1] and JOHN A. FOCHT, JR.,[2] A.M. ASCE.—The primary reason for attempting to determine the soil modulus of laterally loaded piles without a full scale test is to permit a reasonably accurate determination of the pile bending moments, pile deflections, and soil reactions—but principally the bending moments. Prof. Ripperger first questions the existence of "such a thing as a soil modulus," then concedes that "obviously some sort of modulus exists." Very definitely, there is not a unique value of the soil modulus for a given soil as the pile test showed that it varies with depth and pile deflection. As deflection varies with pile size and stiffness, load magnitude, and manner of load application, the modulus will also vary with these factors. The soil modulus exists, therefore, only as a mathematically convenient expression for the ratio of soil reaction to pile deflection, p/y. The term E_s is a symbol for this ratio, and should not be thought of as a property exclusively of the soil.

The tentative correlation coefficient established in this paper does not in itself establish a soil modulus for a given soil, but rather permits establishment of a soil reaction—pile deflection curve (stress-strain curve) for some selected pile size and at a selected depth within that soil. The proper soil modulus for a given problem becomes known only after the problem is fully solved by successive approximations using a difference equation solution or some equally suitable method, and the pile deflection curve is determined.

Mr. Reuss and Prof. Matlock have requested the author's estimate of the effect of inaccuracies in the pile test on the correlation coefficient. To this reasonable question of "what is the possible per cent error in the coefficient?" should be added the question "what degree of accuracy is required of the coefficient?". Mathematical analyses using idealized E_s versus depth relationships shed some light on the second question. For E_s constant with depth and deflection, the pile bending moment, M, varies as the fourth root of E_s.(1) For E_s constant with deflection but linearly increasing with depth, M varies as the fifth root of E_s.(2) To compute M to within a 5 per cent range, E_s could be in error about 20 per cent in the first case and about 25 per cent in the second case. Returning to the first question stated above and considering the accuracy of the correlation coefficient to be affected only by the precision of the computed deflections and reactions for the test pile, the possible error in the value of the coefficient might be as much as 50 per cent. This could permit an error in a computed moment of 10 per cent. The actual error resulting from inaccuracies in the test pile data is believed to be less than 50 per cent, because of the statistical treatment given to the data by the experimenters,(3) and because of partial confirmation given by the

a. Proc. Paper 1081, October, 1956, by Bramlette McClelland and John A. Focht, Jr.

1. President, McClelland Engineers, Inc., Soil & Foundation Consultants, Houston, Tex.

2. Chief Design Engr., McClelland Engineers, Inc., Soil & Foundation Consultants, Houston, Tex.

rational analysis of Prof. Reese. The correlation coefficient, with intelligent use, is therefore believed to be of sufficient accuracy to permit calculation of pile bending moments with an error of less than 10 per cent for loading conditions similar to the test.

In most problems involving laterally loaded piles, the desired product of the analysis is the pile bending moment at or near the ground line. The "significant depth" of soil (questioned by Messrs. Peck, Davisson and Hansen) governing pile behavior has been shown by previous studies[4] to be down to the point of zero deflection. Significant changes in the assumed value of soil modulus below this point will have very little effect on the computed moment and deflection at the ground line. Fig. 13 (taken from a previous paper by the authors) clearly demonstrates this fact. Prof. Reese also found that the upper 15 feet of soil at the test site governs the pile performance even though the embedded pile length is 75 feet. The relative stiffness of the pile-soil system below the point of zero deflection has only limited effect on the deflection curve near the mudline.

Dr. Peck and his associates apparently attach much importance to the irregular and small values of E_s indicated by the test pile data below 15 feet. They developed with considerable discussion a belief which the authors accepted by inspection: that much of this irregularity is due to inaccuracies in the deflection and reaction determinations in a zone where both of these values are of small magnitude. They indicated that adjustment of the pile deflection curve by a minor rotation so as to equalize the depths to zero deflection and zero reaction will produce more reasonable E_s values to a depth of as great as 30 feet. However, they apparently failed to recognize or to acknowledge that this same rotation produced negligible effect on the E_s values above 20-foot depth; and that these latter values are the ones which controlled the pile behavior and on which the correlation coefficient was based. The prime importance of the values at shallow depths and the relative unimportance of the values at greater depths was stated by the authors with a reference cited; Prof. Reese independently arrived at the same conclusion.

The possibility that the effects of the loading sequence may have had marked effect on the results is raised by Dr. Peck and his associates. There are now reasons[5] to believe, if the soil is not stressed to failure, that a small number of load applications will not cause any significant decrease in E_s at a given depth. A total of only 57 load applications, both static and dynamic, were made to the pile. The magnitude of the loads, in general, progressively increased. Some erratic errors could have developed due to the necessity for switching between loads but a procedure of statistical averaging of the basic moment data, as followed by the test sponsors, probably reduced most of these.

Prof. Reese's derivation of 12c as the ultimate soil resistance below the ground surface is substantiated by a value of 11.42 computed by Meyerhoff,[6] both of which compare favorably with the value of 11c computed from Eq. 4. Thereby, Eq. 4 is shown to have theoretical as well as empirical basis.

The zone near the ground line boundary, as Prof. Reese pointed out, is one requiring further study. On Fig. 14, the ratio of ultimate soil resistance, p/b, to the soil shear strength is plotted versus the depth-diameter ratio. The soil resistance was computed from Prof. Reese's Eq. 8 assuming $\theta = 45^o$, $\gamma = 0$, and $k = 0$. Four points computed from the maximum soil stresses resulting from static loads (taken from Fig. 8) and from the average Q_c triaxial shear strength curve on Fig. 6 are plotted on Fig. 14. These data confirm

Prof. Reese's conclusion that the ultimate soil resistance will be less than 11c or 12c within a depth range of about three pile diameters. The authors believe, on the basis of limited evidence, that Eq. 4 may be valid even at shallow depths if the applied loads produce soil reactions less than the maximum resistance. The presence of the groundline boundary does permit failure at a lower strain than if the soil surface were further removed but may not significantly affect the stress-strain curve up to the point of failure.

The emphasis placed by Prof. Reese on thorough exploration of shallow soils for laterally loaded structures is certainly warranted. The most careful sampling and testing techniques should be observed in order to eliminate the effect of variable techniques of sampling and laboratory testing. Messrs. Peck, Davisson, and Hansen conclude that possible variations in sampling and laboratory procedures make application of the tentative correlation unjustified. However, since these variables can be controlled to obtain reproducible results, their conclusion does not seem warranted.

Lateral load tests in which only deflection and rotation of the pile head are measured as suggested by Prof. Reese to obtain soil resistance and pile deflection curves will yield results only as good as the assumption of $E_s = kx$. It appears much more preferable to fully instrument the test pile as was done by Profs. Matlock and Ripperger(7) than to test uninstrumented piles. The remarkable degree of precision developed by Profs. Matlock and Ripperger in their instrumentation of a laterally loaded test pile constitutes a highly desirable precedent for future pile tests. In their tests, supplemental measurements of deflection and slope at selected points and of the applied load permitted minor adjustments to reduce the errors introduced by such effects as slight rotation of the pile.

The ingenious diagrammatic representation of the pile soil system given by Prof. Ripperger shows clearly the complexity of the problem. In his consideration of a thin horizontal slide from the pile-soil system, Prof. Ripperger states that "stress distribution in this slide . . . . is the same as for a concentrated load applied to a point in an infinite plate . . . <u>except for differences in the vicinity of the point of load application.</u>" He then developes that deflection theoretically should be infinite for all loads, which means that the "soil modulus would have no meaning and no unique value other than zero." Although Prof. Ripperger mentions several factors which may explain why actual deflections are not infinite, the authors believe that he overlooked the prime factor which is the exception in his assumed load distribution underscored in the above quotation. As Timoshenko(8) has previously pointed out, the infinite deflection at the point of application of a concentrated load may be disregarded as only theoretical, and a finite distributed load on the edge of an infinite plate or on the surface of a semi-infinite mass will create a finite displacement. Therefore, the prime factor producing finite deflection is the finite area of load application rather than the other factors mentioned. Thus, with finite displacements having a fundamental basis, there should be finite values of the soil modulus which will vary with depth, deflection, pile size, pile stiffness, pile length, and soil type; the influence of these variables on the deflection can be reasonably estimated by following the suggested method for applying the proposed correlation.

Mr. Reuss's discussion is most welcome as it is he who first proposed the comparison of field and laboratory stress-strain curves on logarithmic paper(9) as utilized in this paper. The problem of a factor of safety in situations involving laterally loaded piles is a significant one which has not been

solved. There are several ways in which a laterally loaded pile structure could fail.

1. The pile could move through the soil failing the soil for the full length of the pile;
2. The pile could fail structurally due to excessive bending stresses; or
3. The deflection, or rotation, of the pile could be detrimental to the utilization of the structure.

The first and third modes of failure will probably occur in piles which are relatively stiff with short penetrations. The second type of failure will probably develop in piles of relatively great penetration with low stiffness. Inasmuch as the technique of predicting at what load a failure will occur and what type of failure will develop is only tentative, a relatively large factor of safety is desirable. The inclusion of a factor of safety by increasing the design load as suggested by Mr. Reuss is one satisfactory way of handling the problem. In the case of laterally loaded piles for offshore structures, however determination of the loads to be imposed is also subject to diverse errors. The probability of the combination of conditions to produce the maximum load is another uncertainty. Therefore, it has been necessary for offshore structures to use design loads without application of a safety factor and accept a calculated risk.

The possibility of a linear (or elastic) deformation followed by a non-linear (or plastic) deformation as the load increases is presented by Prof. Ripperger and Mr. Reuss. Mr. Reuss's suggestion that the deflection be limited to within the elastic range to avoid progressive failure has merit as Gaul's data(5) shows that within the elastic range, repeated loads do not cause progressive failure. There are some indications of elastic action under the dynamic loads as the stress-strain curves on Fig. 8 do have an initial slope of approximately 45°. However, there is no indication of linearity for the static loads in this test, which indicates that the linear range, if any, was small. Therefore, designing for the limitation suggested would limit a static lateral load to almost zero. For overconsolidated soils, a wider range of linear stress vs. strain could be expected.

Mr. Lundgren's discussion amplifies a number of the pertinent assumptions, limitations and results. It seems reasonable to us also to expect a soil reaction greater than zero at 3-foot depth even for a 100-kip load. In the case of the test pile, the effects of cyclic loads and remolding effects could have caused the reaction to be truly zero. It is also possible that the true reaction was a small finite value but due to minor errors in moment observations, the computed reaction is zero in error.

The effect of a large number of transient loads, particularly reversing ones, might tend to increase the strength of the soil as suggested by Mr. Lundgren. But substantial deflections of the soil will result during the process so that the effect should be for the soil modulus to decrease with repeated loads if the deflections are in the plastic range. That is, the deflections of a pile probably will increase as the lateral loads are repeated. There may even be a tendency towards progressive failure. Therefore, more study of the effects of repeated and cyclic loads is required before any definite conclusions can be drawn.

The strain value of 1% used to develop the plot of Fig. 9 was chosen for purposes of illustration only. Similar plots for strains of 0.5% to 3% would

yield correlation factors differing only slightly from 5.5. Other tests to extend, verify, or disprove the tentative correlation are definitely and urgently needed.

In summary, any design procedure based upon a single test, or even one test series covering a significant load range, can be compared to extrapolation from a single point. General considerations can provide guides for the extrapolation but the procedure must be considered tentative and subject to even major revision until further data is secured. The discussions of this paper, for the most part, indicate that there was a definite need for a starting point whereby solutions for laterally loaded piles may be obtained. Messrs. Peck, Davisson and Hansen pointed out that "E_s must decrease markedly with increasing deflection," and that "any theory based on the assumption that E_s is independent of deflection is likely to be seriously in error." The tentative correlation and the proposed method for its application to practical problems is the only procedure that has been advanced for estimating the decrease of E_s with increasing deflection. With the application of careful thought and analysis as displayed in several discussions and with additional fully instrumented lateral load tests, a more precise correlation substantiated by well documented data can be developed by which the stress-strain relationships for laterally loaded piles can be accurately predicted.

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8. Timoshenko, S. P., "Theory of Elasticity," McGraw-Hill, 1934, p. 88 & p. 333.

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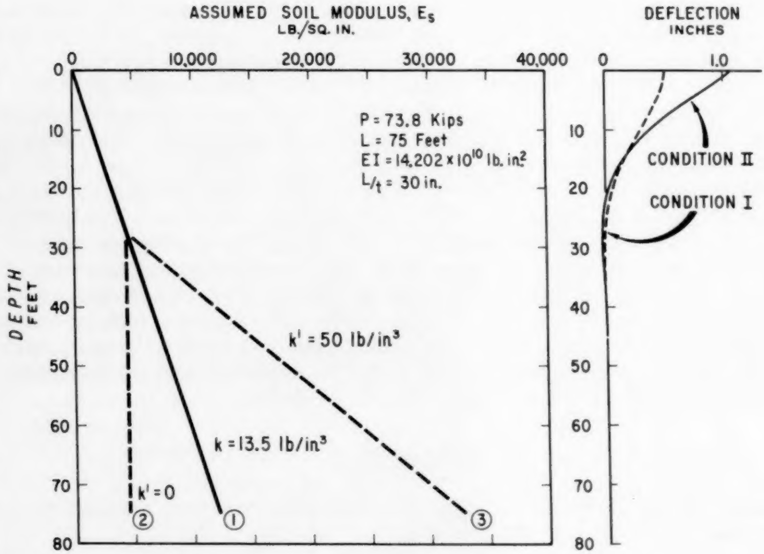
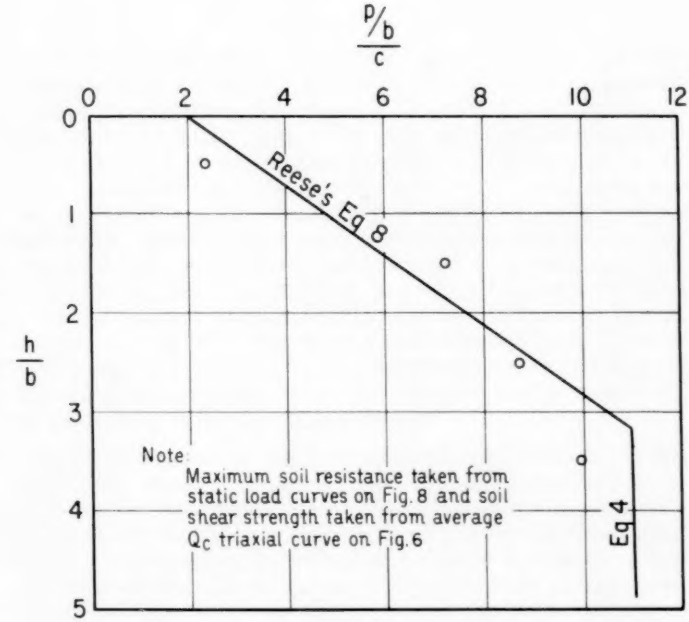


	ASSUMED E_s	DEFLECTION AT GROUND LINE INCHES	MOMENT AT GROUND LINE FT.-KIPS
CONDITION I	1	0.508	579
SLOPE = 0	2	0.508	579
	3	0.507	578
CONDITION II	1	1.073	182
SLOPE = -0.006	2	1.073	182
	3	1.071	181

EFFECT OF CHANGE IN k

Fig. 13



RATIO OF MAXIMUM SOIL RESISTANCE TO SOIL SHEAR STRENGTH

Fig. 14

MOISTURE CONDITIONS UNDER FLEXIBLE AIRFIELD PAVEMENTS[a]

Closure by J. F. Redus

J. F. REDUS,* A.M. ASCE.—The author appreciates the interesting information presented by Messrs. Guinnee and Thomas concerning moisture conditions under rigid highway pavements.

Several items have developed in the study of flexible airfield pavements since the preparation of the original paper and are of interest in connection with the discussion of subgrade moisture contents under flexible highway pavements in Missouri. Colman fibre-glass moisture cells were installed in pavement, base course, and subgrade of a special test section at the WES and on one airfield. An exact calibration of the cell readings (electrical resistance) in terms of percentage of moisture could not be made for the highly compacted materials in modern military fields, but the cells indicated moisture-content changes and temperatures very reliably. A program has been developed using the cells to determine temperature and moisture change, and direct sampling procedures to determine actual moisture-content values. Several interesting facts have been revealed:

1. Appreciable moisture-content increases were indicated in the base course and subgrade surface about a foot from the pavement edge after several days of intermittent rain. The amount of increase was greatest opposite a point where water ponded to a depth of about 1 in. on the shoulder at the pavement edge. Corresponding decreases in moisture content were indicated in two or three days after the rain. No moisture-content changes were indicated at depth of 6 in. below the subgrade surface at 1 ft. from the pavement edge or in the base or subgrade at 10 ft. from the pavement edge.

2. Very small daily moisture-content changes occurred in the base course and subgrade surface as a result of changes in the thermal gradient.

These statements indicate that on the test section appreciable moisture changes occurred in the materials just under the pavement and near the edge, but not much below the subgrade surface or as much as 10 ft. from the edge. One example of an opposing situation can be cited, however. About 3 ft. of fill was placed over a natural material with a moisture content of about 3 per cent to a width of about 75 ft. Approximately a year later, the moisture content of the natural material on the center line had increased to about 12 per cent (optimum), while that of the layers above had changed very little. The water table is at a considerable depth.

a. Proc. Paper 1159, January, 1957, by J. F. Redus.

* Engr., Flexible Pavement Branch, Soils Div., U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

FRICTIONAL RESISTANCE OF STEEL H-PILING IN CLAY[a]

Discussion by Robert L. Mc Neill and Raymond Lundgren

ROBERT L. MC NEILL,[1] J.M. ASCE and RAYMOND LUNDGREN,[2] A.M. ASCE.—Mr. Vey has presented an interesting and enlightening paper. The action of steel H-piles is little understood; most research has been centered around cylindrical piles, probably because of the relative simplicity of approach. H-piles, on the other hand, have rather gross sectional discontinuities which may make their analysis more difficult and less certain.

In normal practice, one analyzes a cylindrical friction pile by integrating some soil shear strength along the length of the pile. For cylindrical piles, the method and the mathematics are probably correct (neglecting point resistance), and the only variable is the choice of the soil shear strength. For H-piles, however, the discontinuous section shape surely must introduce some error into the method because of boundary conditions. There are but a few indications in the literature on how the frictional resistance of H-piles is computed in practice. Some manufacturers' publications[3] consider the whole outside perimeter of the H-pile and compute the average "sustaining value" of the pile by dividing the safe load estimated from the results of actual loading test by the total pile contact surface. In such computations, however, high "sustaining values" result mostly when a pile is driven to a layer offering a high point resistance. Thus the point resistance as well as the soil shear and the frictional resistance are lumped into the one parameter, "Sustaining Value." To the practicing Engineer, a "Sustaining Value" that is not clearly related to soil properties is of little significance.

Mr. Vey states that the shear strength of the soil applied to the effective section of the pile formed by a line connecting the outside edges of the flanges gives good agreement with observed data. This is no minor assumption; for the pile considered, the flanges would be capable of contributing up to 50% of the total capacity if the soil shear strength were mobilized around the total pile perimeter. The piles were load-tested about three months after they were driven;[4] this is ample time for the most significant consolidation and thixotropic effects to have occurred. One would therefore expect the piles to be near their maximum capacity; and yet, in order to make the observed results fit the available data, one must assume that only one-half the available perimeter contributes to the capacity of the pile.

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a. Proc. Paper 1160, January, 1957, by Eben Vey, A.M. ASCE.

1. Staff Engr., Woodward, Clyde and Associates, Oakland, Calif.
2. Partner and Principal Engr., Woodward, Clyde and Associates, Oakland, Calif.
3. For example; "Steel H-Beam Bearing Piles," United States Steel Corporation, no date given (published prior to issuance of the paper under discussion).
4. Janes, R. L., Assistant Manager, Armour Research Foundation, letter of May 23, 1957.

It is inconceivable that the flange areas contribute nothing to the bearing capacity. If failure is to take place in the soil along a line connecting the tips of the flanges, then an appreciable amount of soil-on-steel friction must be mobilized on the surface of the web plus the inside flanges. Since, for all practical purposes, the length of a line connecting the flange tips is one-half the length of the web plus the inside flanges, the soil-on-steel friction must be at least one-half the shear strength of the soil for failure to occur in the soil; otherwise, failure will occur at the soil-pile interface around the entire pile perimeter. The outside flanges comprise one-half of the external perimeter; therefore, for failure to occur in the soil, the outside flanges must contribute at least 33 percent of the total pile capacity. As was previously pointed out, the maximum flange contribution may be up to 50 percent.

An analysis of a recent paper by Mansur and Kaufman[5] seems to indicate that the hypothesis advanced by Mr. Vey may be quite in error when applied to other situations. In the Mansur-Kaufman paper, H-piles were driven through a silt deposit; the strength properties of this material were given as $\phi = 28^{o}$ and C = 0.1 ton per square foot.[6] Mechanical strain-measuring devices were affixed to the piles, and strains were measured along the length of the pile as load tests were conducted. From the results of these measurements, it is possible to calculate the load transferred to the soil within each length increment being considered. Having this load, the calculated soil shear resistance depends only on the effective perimeter assumed. Figure 1 has been prepared from the data given by Mansur and Kaufman.[7] On the figure are shown the soil shear strength-depth relationship, as well as two curves of calculated soil shear strength from the pile load data, assuming the two effective perimeters under consideration. Curve (A) was calculated by assuming that the entire outside perimeter of the H-pile is effective in mobilizing soil shear strength; within the limitations of the idealized soil strength properties given in the paper, there are some grounds for optimism concerning the validity of this curve for the case considered. Curve (B) was calculated by assuming that the effective perimeter is bounded only by lines connecting the ends of the flanges; this curve is seen to deviate appreciably from the known soil strength characteristics. Therefore, comparing the Vey and the Mansur-Kaufman data, one might infer that the final answer is not yet in hand. It is possible that computing H-pile capacities from soil skin friction alone is an incorrect approach. Apparently other factors such as soil-on-steel friction and boundary conditions should be included in any rational analysis of H-pile capacities.

It should also be pointed out that the soil considered by Mr. Vey was a rather insensitive clay. In more sensitive clays, where remolding may be more severe and where reconsolidation and thixotropy may play an important part in determining the true soil strength, the effect of disturbance due to driving could be considerable.

Although this discussion is at variance with Mr. Vey's paper on some points, this is in no way a criticism of the work on the paper. The analysis of the data and the paper were admirably done by Mr. Vey and will be a lasting contribution to the profession.

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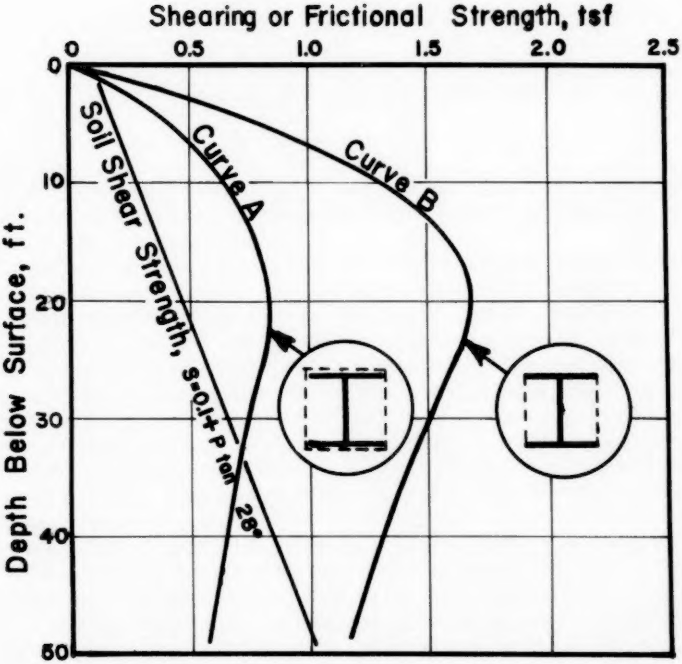
6. Ibid, p. 1079-3.
7. Ibid, p. 1079-13, 1079-15.



NOTES

- Curve A assumes full shear perimeter.
- Curve B assumes half shear perimeter.
- Water table assmed at surface, soil density assumed 130pcf.
- Data extracted from Mansur & Kaufman, ref. 5.

FIGURE 1: SHEAR STRENGTH TO RESIST MEASURED LOAD AT VARIOUS DEPTHS ON STEEL H-PILES.

RELATIVE DENSITY AND STRENGTH OF SANDS[a]

Discussion by Yoshichika Nishida

YOSHICHIKA NISHIDA.[1]—The author presents some new relationships of worth on relative density, to which the writer sends his respects. The writer wishes to pose a few questions.

The author explains the fact that the relative density of soil bears an inverse relationship with their particle size, after compressibilities. The writer understands that the relative density is that of the soil's natural state in the ground. If so, the natural void ratio e in the relative density D_r must be influenced by the effective pressure imposed on soils. In the comparison of the relative density between the coarse sands and the fine sands was it done under the same overburden effective pressure? Since the relative density is $(e_{max} - e)/(e_{max} - e_{min})$, it does not become small when $(e_{max} - e_{min})$ is small, even if e has a close value to e_{max}, as the author states for the coarse sands and gravels. The writer wishes to know the influence of the depth to the penetration resistance.

A general trend of increasing initial modulus with increasing relative density (accordingly with void decrease) was also obtained by the writer's experiments for clay and silty soils, in which the initial modulus was constant if the void ratio of the soils were not changed although the confining pressure were not equal. According to Boussinesq the elastic modulus is assumed to be proportional to the confining pressure for sands, and this is in experimental noted by the vibration tests. In the author's experiments further study is desirable on the relationship between the confining pressure and the relative density.

More explanations are desired on the mean diameter and the standard deviation which are not generally familiar to soil engineers. How is the mean diameter defined?

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a. Proc. Paper 1161, January, 1957, by T. H. Wu.

1. Instructor of Civ. Eng., Kanazawa Univ., Kanazawa-shi, Japan.

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FIELD EXPERIENCES WITH CHEMICAL GROUTING[a]

Discussion by Judson P. Elston and Glebe A. Kravetz

JUDSON P. ELSTON,* M. ASCE.—This paper is certainly a contribution to the phase of foundation treatment known as chemical grouting. As the authors indicate, the development of chemicals in an effort to find a means of successfully solidifying or consolidating sands and sandy silts has been in progress since the early 1900's. It is felt, however, that much remains to be done, both in the laboratory and under actual field conditions before it becomes reliable from the economic and engineering point of view. The writers' experiences with chemical grouts, both in the laboratory and in the field, go back to 1947 while associated with the Office of the Chief Engineer, U. S. Bureau of Reclamation, Denver, Colorado.

A. It is believed that the first instance of attempted chemical solidification of foundation soils by the Bureau of Reclamation was for the footings for columns for a warehouse at Davis Dam, Arizona. The engineer-in-charge of this work in the field was Bert C. Wilkas. The general specifications were based on Federal Housing Administration requirements issued in 1943. Sodium silicate and calcium chloride were the basic solutions used. Six-inch auger holes were drilled four feet deep, backfilled with pea-gravel after which the sodium silicate was gravity-fed until the soil refused further chemical and the level remained two inches above the bottom of the trench. As soon as the free surface of liquid settled below the floor of the trench, additional liquid was added. After absorption in the soil for a period of 24 hours had taken place, calcium chloride with a small quantity of sodium chloride buffer was fed into the soil in a similar manner to the first chemical. While somewhat primitive methods of mixing and injecting were used, the results were quite gratifying. Loading tests conducted on footings both before and after the chemical treatment indicated solidification of soils had been accomplished. The purpose of the program which was to eliminate settlement in the foundations for the warehouse, in other words to sustain the design loadings, was accomplished.

B. In 1949, the laboratory work was reviewed and assistance was given to conducting field trials of the silicate grouts in the Navajo sandstone at the Glen Canyon Damsite in Arizona. The field phase of this work involved living at the Glen Canyon site for a period of four weeks. The interesting part of this work was the conditions under which it was carried out. All material, equipment, fuel, tents, tanks, drills, pumps, etc., had to be brought in by boat 15 miles up the Colorado River from Lees Ferry. When one considers the problems and difficulties involved in transporting materials, equipment and men 15 miles against current, bars, rapids, and climatic conditions with two

a. Proc. Paper 1204, April, 1957, by Milos Polivka, Leslie P. Witte and John P. Gnaedinger.

* Asst. to Constr. Mgr.—Power Area, Uhl, Hall & Rich,—Engrs., St. Lawrence Power Project, Massena, New York.

outboard boats into one of the most isolated areas in the United States, it is understandable why, regardless of results, the operation was considered a success.

Water test data from four drill holes gave an average permeability (K) as 64 feet per year with individual values at various pressures ranging from 21 to 172 feet per year. Grout holes were drilled in an excavated bench area with a few holes being drilled both upstream and downstream from this area along the base of the cliff which will form one of the abutments for the dam. Two sizes of holes were used—BX diamond drill holes and 2-inch diameter percussion holes. Where water was used ahead of the chemical grouting, the holes were flushed with water under pressure and some of the holes were scrubbed mechanically. Other holes drilled dry were flushed with air and water. Pumping pressures ranged from 5 to 150 pounds per square inch with consistent pumping at 80 pounds per square inch. Very little of the chemical mixtures penetrated the sandstone. One interesting observation was the effect of the chemicals on the metal parts of the plant. Field mixes were agitated in the mixer and circulated through a simplex pump for several hours to observe the action of the chemicals on various pieces of equipment. Visual reaction between the grouts and metals with which they were in contact was absent; however, the chemical reaction did tend to deposit a fine film in the drums, pipe lines, fittings, and in the pump, which could not be removed except by sandblasting or tedious scraping. This film became very noticeable in operation by the way it affected opening and closing of valves, and the working of the valves and pistons in the pump.

Conclusions reached from the work at Glen Canyon were that the viscosity of the so-called silicate grouts always had to be higher than water. The closest specific gravity of a silicate grout was 1.03 but the shrinkage factor upon drying was 97 percent. A silicate grout with a low dry factor had to have a viscosity of near 1.30. This meant that in many foundations in which water could be pumped, a long-life stable silicate grout could not be injected because the mixture was too thick to penetrate the fine material. Another problem was that as the specific gravity approached 1.0, the difficulties to controlling the mix and setting time multiplied at an alarming rate. It is felt that this work again illustrates the need for treating each foundation problem as a separate entity in itself. The actual water, hardness or softness (from the salt or acid point of view), temperatures at the time of mixing, and a multitude of other factors all aside from the characteristics of the foundation itself are extremely important. Silicate-type groups do hold promise in some types of soils. To be successful, additional field research at the site seems indicated.

C. The writer was extremely interested in the description of the use of chrome-lignins at Heart Butte Dam because of his very intimate association with this work. In fact, the writer evaluated the available literature on the chrome-lignins and the work done in the laboratory which led to the recommendation that chrome-lignin be experimented with in the field under actual site and operating conditions. In November, 1953, the writer travelled to Bismarck, North Dakota, and to Heart Butte Dam and, with the valuable assistance of R. W. Burrows and field personnel, personally supervised the chrome-lignin grouting field trials. One is not in complete agreement with Mr. Witte's statement that "No appreciable amount of cement could be injected. . . ." It is recalled that during the summer of 1951, an intensive

cement grouting program was carried out at Heart Butte. Also, that some of the leaks were completely controlled and grouted off and that others were greatly reduced. One is of the opinion that the program in general, was successful considering the difficulties usually encountered in grouting sands with neat cement grouts.

1. It is recalled that the selection of the chrome-lignin mixture used at the site was based on the following procedure: The proportion of sodium dichromate and sulphuric acid to lignin was varied until a gel was produced which became insoluble in water after the initial setting and hardening period. The proportion of the catalyst, ferric chloride was increased until the setting time was reduced to a minimum commensurate with the successful mixing and injection of grout, about 1 to 2 hours. Water was then added until the viscosity was reduced to a point where excessive pressure would not be developed in pumping the grout through the equipment and hoses and to obtain better penetration into the sand-like formation. The grout mixture selected from these tests contained almost twice as much dichromate as the laboratory tests indicated would be necessary but required no acid because of its high acidity.

2. At the job, the grout mixture was prepared by batching by volume the previously prepared solutions of the chemicals. It was found that temperature measurements of the grout gave indications of the accuracy of the batching procedure and also revealed when the grout was in danger of setting up in the equipment. The initial temperatures of the lignin remained at about 44° F. After the addition of the other chemicals, it was found that if the temperature rose to about 55° F. in 5 minutes and to about 60° F. in 15 minutes, the grout would remain at pumping consistency for an hour or slightly more. A temperature rise to 70° F. indicated that the mixture was about ready to gel and that the remaining grout should be either quickly injected or wasted, and the work of flushing and washing out all equipment, tanks, and lines commenced. This was usually a reliable 5 to 8 minute warning period as far as grout setting up in the pump was concerned.

3. In conclusion, the work conducted at Heart Butte Dam outlet tunnel can be summed up as follows:

a. That chrome-lignins could be easily prepared and pumped using regular cement grout plant equipment.

b. That chrome-lignins certainly penetrated smaller voids in sands than cements or any grouts containing solids, including colloidal grouts.

c. That chrome-lignins held promise of penetrating any material in which water could be injected (specific gravity could be adjusted to 1.0) and in many aspects were a more promising chemical grout than the silicate type grouts, being a true chemical solution, particularly where strength of consolidated material is not a primary requirement. They form an insoluble, elastic, adhesive material with a hardness similar to hard rubber automobile tires.

d. That, as is the case with all of the chemical grouts, experienced to date, the chrome-lignins need on-the-site laboratory tests under the identical field conditions to which they will later be subjected.

e. The plastics or resin-type chemical grouts, the writer believes, may eventually hold more promise than either the silicate or chrome-lignins. It has been proven that they can be controlled and injected with but little difficulty. The shrinkage factor is non-existent and the resultant material is of high strength. Mr. Gnaedinger's experiences are extremely interesting. It was a little surprising to hear that the costs of the work were only one-third what they would have been with other types of chemical injection. It had been understood, perhaps erroneously, that from a cost point of view the resins were prohibitive. Certainly, if resin plastics can be bought, handled, mixed and pumped at a cost close to that involved with the silicates or the chrome-lignins, they should be given serious consideration for all foundation problems in soils when chemical grouting becomes one of the proposed methods of foundation treatment.

GLEBE A. KRAVETZ,[1] J.M. ASCE.—When pressure grouting of a soil foundation is considered, the first problem usually is to determine if the soil can be grouted. Classifications exist which indicate the type of mixture that can be successfully grouted into different types of soils. These soils are classified according to their grain size distribution. If mixtures containing cement, clay or asphalt emulsion are considered, the following equation is used also

$$\frac{D_{15}}{D_{85}} > N$$

In this equation, D_{15} is the 15% passing size of the soil.
D_{85} is the 85% passing size of the grout.
N is a dimensionless number, which may vary between 5 and 20.

This formula and the classifications are simple to use, because the grain size of a soil is easily determined. However, they are only indicative, inasmuch as grain size is only one of the factors affecting the voids of a soil.

Mr. Polivka's statement that "chemical grout. . . can be injected into any soil into which water can be injected" represents a new approach. It is strictly correct but implies that methods to determine whether water can be injected into a soil exist.

It is believed that a more accurate way of determining the groutability of a soil would be to use in classifications or formulae to be established the soil coefficient of permeability K, rather than the grain size. Permeability reflects both the ratio and size of a soil voids. As a soil property, it has been most extensively studied and laboratory and field test procedures are well known. The use of K seems even more justified if one realizes that laboratory and field grouting tests are nothing but permeability tests with a different purpose.

Laboratory grouting tests, as further means to determine if a soil can be grouted, are too often inconclusive. There are many reasons for it, but most of the time it is because the soil as placed in the cylinder is not in the same condition than in the ground, or because the test set-up does not allow a

1. Soil and Foundation Engr. Tippetts-Abbett-McCarthy-Stratton, Engrs., New York, N. Y.

uniform penetration of the grout. Reproduction of the natural conditions of the soil, accurate testing procedures and careful interpretation of the results are all important, as illustrated next.

In the test performed in connection with the construction of a 12 ft. diameter tunnel in Detroit, compacting the sand in the 55-gallon drum before the grouting test was certainly not giving it the same void ratio than in the tunnel heading where it was under excess hydrostatic pressure. In the laboratory, samples to be grouted can be subjected to excess hydrostatic pressure by the very same method which is used to demonstrate the hydraulic conditions associated with boiling of sand.

Very often, the soil in the cylinder is placed right against the weep hole(s) or outlet(s). As a result, a very sharp drop of pressure is created within the sample. In the field, this condition exists only during surface grouting at which time grout always tends to come to the surface along the injection needles. In deeper zones because of the lateral confinement, the drop of pressure is gradual and the grout penetration is more uniform. This latter condition can be reproduced in the laboratory by leaving a space between the top of the sample and the top of the cylinder where the outlet is located. This space is filled with water, inasmuch as the sample is usually saturated before the grouting test. The outlet is equipped with a valve and a bourdon gage is installed at the top of the cylinder. Thus, the effective pressure, which is the difference between the pressure applied and the pressure at the top of the cylinder, can be controlled at all times. Both the flows of the injected grout and the displaced water are measured, and the water is analyzed. It is therefore possible to record the progress of the grouting, to find out, when the grout appears at the outlet, how much water remains in the sample and also if the grout mixes with the ground water when it displaces it forward.

Without this checking, results may be at times, quite baffling as was the case in one grouting study: cylinders of soil were solidified in an apparatus similar to the one represented on Figure 2 of the paper and permeability test results performed on them were both erratic and most disappointing. Eventually (and accidentally) it was discovered that the grout penetrated the sample for no more than one-half to one inch before escaping through the outlet, so that after the grout had set a crust was formed over the ungrouted sand.

In the paragraph on the one-shot method, the sodium silicate-sodium bicarbonate gel is described as temporary and the sodium silicate-hydrochloric acid and copper sulfate gel as permanent. It is the writer's belief that wherever water exists in the ground, no gel can last forever: it is only a question of time, until the gel is leeched away. Also, the authors, referring to the latter gel, state that "(it) is more complex to prepare the more expensive." In the writer's experience, it is not so. Copper sulphate comes usually as a powder, and its mixing with hydrochloric acid and water in pre-determined proportions* does not require any specific sequence nor precaution. By adding copper sulphate, the quantity of the hydrochloric acid which would be otherwise required by 25%, and because the acid is diluted there is less water in the resulting gel, thus making it stronger. Preparation of any one-shot sodium silicate-reagent mixture does not present any difficulty if the following steps are followed:

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* For instance, 100 milliliter of hydrochloric acid, 900 milliliter of water, 5 grams of copper sulphate.

1. Both sodium silicate and reagent are diluted with water and thoroughly stirred before being mixed together.

2. Reagent is always poured into the sodium silicate, never the opposite.

3. Sodium silicate must be thoroughly agitated while the reagent is being added.

In spite of all the laboratory testing and careful preparation of grout, the final success of a grouting job will always depend on the selection of the right field methods which requires from the engineer a thorough understanding of the ground conditions and most especially of the ground water conditions.

For instance, depending on the location of the grout holes and their grouting sequence, excess water pressure or water circulation can either help or limit the diffusion of the grout in the ground. This can be illustrated by discussing (without any thought of criticism inasmuch as the field conditions are not known to the writer) the grouting of the railroad underpass at LaGrande, Oregon. It may be seen from the description given that the water under the pavement was definitely confining the diffusion of the grout. As a result, it is believed that only the sub-base immediately surrounding the injection holes was solidified. Consequently, it may be feared that cracks and spalling may eventually develop in areas of the pavement where the sub-base was not solidified. Assuming that the grouting of the sub-base was possible from behind the walls, through the railroad embankment, and that a one-shot mixture could be used, it is believed that this method would be preferable. In this way, the water would carry the grout towards the cracks and relief holes drilled in the pavement. As the grout would start to appear at the surface, holes and cracks would be gradually plugged or caulked, thus forcing the grout to spread over a bigger area.

Under special circumstances, water conditions can be modified, in order to facilitate the grout circulation. This can be done by using well-points or deep pumps to create local water circulation and/or excess hydrostatic pressure.

It is believed also that at one time "developing" of granular deposits, by removing the fines through heavy pumping from deep holes was considered in an attempt to consolidate the ground with cement grout. It is not known if these attempts were ever successful, but they might work if chemical grouts were considered, inasmuch as much smaller fines need to be removed.

Soil pressure grouting, like other soil and foundation work, requires from the Engineer a vivid imagination, but restrained by technical knowledge and practical experience. Through research work and field experience, it is becoming more of a technique and less of an art. In this effort, Messrs. Polivka, Witte, and Gnaedinger's paper is a most valuable contribution.

COMPACTING EARTH DAMS WITH HEAVY TAMPING ROLLERS[a]

Discussion by J. D. Hodgson and C. Y. Li

J. D. HODGSON.—In Paper No. 862 published in SM 1 January, 1956, C. Y. Li outlined the basis principles of compaction of soils and the writer feels that if this concept were applied to the data given by the author it will be seen that on a number of occasions large amounts of energy were wasted by obtaining densities in excess of the design density. The writer believes it is normal Bureau of Reclamation practice to use shear strength figures obtained on samples prepared to optimum conditions for the design of embankments and during construction the aim is to place the soil well below the optimum moisture content. It is noticed that the average densities obtained rarely fall below the optimum although the moisture contents are mainly substantially below optimum. This indicates that the optimum moisture content for the rollers used is probably considerably lower than that obtained in the laboratory. The figures given by the author do not permit of an estimate being made of the number of occasions on which excessive foot pressures caused more passes to be used than would have been required by properly adjusted rollers.

It is possible that other types of rollers would have permitted deeper layers to be used with a consequent lessening of the energy/cu.ft. applied to the soil. Small variations (up to 10%) in density from the top to the bottom of a layer are of little significance when the possible variations due to improper rolling, soil variations and other causes are considered. The overall consolidation of the bank would even out such variations almost by the time construction had been completed, except in highly impermeable soils.

Table 1 is a list of the variables in the problem of compaction listed under two headings. Variations of any variable in one heading can affect one or more under the other heading. They are listed in order starting with those which are normally fixed by nature or plant availability thence through increasing order of control by the engineer. One of the most important features of the various types of rollers is the different moisture contents at which a particular soil can be satisfactorily compacted. This feature is the most powerful weapon available to the engineer to reduce earth dam construction costs.

On small projects the most economical design will be that which uses available plant on the soil as excavated to produce reasonable strength. In practice small amounts of water can be added and better types of material selected for use in certain zones without contributing greatly to the cost of placing.

On large projects where small increases in strength may mean large savings in material more attention should be given to the selection of the roller in order that the maximum strength can be obtained with a minimum expenditure of energy in compaction. A point would be reached where the saving in material caused by the increase in strength was balanced by the cost of the increased energy required in compaction. Increases in compaction and strength may not be obtainable at the natural moisture contents of the soil and further

a. Proc. Paper 1205, April, 1957, by Jack W. Hilf.
1. G. E. Mech. E., A.M.I.E. Aust.

TABLE I.

Roller	Product
Type	Type
Size	Moisture content
Weight	Thickness of layers
Bearing pressure	Number of passes
	Strength
	Air voids
	Density
	Shape of Dam

expense may be incurred in reducing or increasing the moisture content of the soil.

More use could be made of the compacting effort of traffic on fills if definite information were available on the problem of bonding layers. In granular materials there seems to be no obvious reason for supposing that the bond of loose material on a hard moist smooth surface is any less effective in producing intergranular reaction than the bond of loose material on disturbed soil. The grain sizes are the same in each case and after compaction the contact area will be approximately the same as on any other plane taken through the compacted soil. Whether this is also true for clay soils is doubtful but provides an interesting avenue for research.

At Adaminaby Dam in the Snowy Mountains Hydro-Electric Scheme examination of excavations in the bank show that each layer can be distinguished and the dividing line is straight and approximately parallel to the surface. The material varies from SC to SF clayey using the modified Casagrande classification system. This dam is being compacted by sheepsfoot rollers and all smooth surfaces are ripped before fresh material is placed. This indicates that the sheepsfoot rollers are producing stratification and that the ripping of compacted soil is merely wasting energy. If this is so, one of the reasons quoted by the author for continuing to use sheepsfoot rollers disappears.

The author states that the dams have been compacted to a state in which a high strength has been obtained without causing high construction pore pressures. In Table II are listed the air voids for as constructed and optimum conditions for the dams listed by the author. In most cases the air voids are such as would ensure no serious rise in construction pore pressures. In 9 cases, Green Mountain, Deerfield, Angostura, Heart Butte, Anderson Ranch, Granby, Boysen, Willow Creek and Vermejo, the writer feels that the materials were over compacted for the moisture contents used. In only one of these was the average fill moisture content at or above the laboratory optimum. This indicates that a more reliable method of field control might be the use of

TABLE II.

NO.	DAM	PERCENTAGE AIR VOIDS. FILL	OPTIMUM
1.	Deer Creek Zones 1 & 2	6.9	5.7
2.	Green Mountain	4.4	3.9
3.	Shadow Mountain	6.5	3.8
4.	Deerfield	3.7	2.9
5.	Jackson Gulch Zone 1	10.2	6.4
6.	Long Lake	9.8	6.0
7.	Angostura	3.3	3.1
8.	Horsetooth	12.3	10.7
9.	Soldier Canyon	12.6	6.4
10.	Dixon Canyon	13.8	6.4
11.	Spring Canyon	13.3	7.0
12.	Heart Butte	3.7	1.7
13.	Olympus	10.6	5.0
14.	O'Sullivan	10.7	8.9
15.	South Coulee	9.1	3.8
16.	Anderson Ranch	2.9	2.9
17.	Davis	6.6	4.3
18.	Dickinson	7.4	3.3
19.	Enders	7.2	5.3
20.	Granby	8.3	4.3
21.	Medicine Creek	8.3	4.3
	Bonny		
22.	North Borrow	8.3	9.1
23.	South Borrow	7.9	6.5

NO.	DAM	PERCENTAGE AIR VOIDS.	
		FILL	OPTIMUM
24.	Cedar Bluff	8.2	5.1
25.	North Coulee	12.5	7.0
26.	Shadehill	6.5	2.8
27.	Big Sandy	4.8	5.6
	Boysen		
28.	Borrow K	6.4	5.8
29.	Borrow B	3.6	3.7
30.	Borrow B-1	9.5	5.0
31.	Carter Lake 1, 2 & 3	7.5	6.7
32.	Keyhole	7.5	5.8
33.	Lauro	7.6	6.0
34.	Platoro	8.0	3.3
35.	Rattlesnake	7.6	5.8
36.	Cachuma	8.9	4.5
37.	Flatiron	6.5	5.9
	Glen Anne		
38.	Zone 1	7.4	5.9
39.	Zone 2	10.0	6.1
40.	Jamestown	7.5	6.5
41.	Willow Creek	4.4	3.5
	Trenton		
42.	Foundation	7.7	6.0
43.	Completion	8.8	6.5
44.	Vermejo 2, 3 & stubble	3.6	2.7

percentage air voids which is independent of optimum but is influenced by dry density and moisture content.

The writer would be very appreciative if the author could advise whether or not abnormal construction pore pressures developed in the 9 dams named.

The author states that rollers other than sheepsfoot have been tried on Bureau dams but does not give any information on the periods for which they were tried or the results obtained. It is hoped that this information will be provided in a future paper as no adequate basis of comparison is available to judge the claim made by the author that sheepsfoot rollers of the type used by the Bureau are more satisfactory than any other. The work carried out by the Road Research Laboratory in England shows that no one type of roller is universally satisfactory.

In conclusion the writer feels that too little attention has been given to the flexibility available in the various methods of compaction and to the economics which can be effected in the construction of dams by relating design requirements to the natural soil conditions and the selection of rollers.

C. Y. LI,* A.M. ASCE.—The publication of the analysis of a very large amount of data on compacting earth dams is very interesting and should be welcomed. Much valuable information on the subject is given. The treatment of the result of test section at Cachuma Dam is an excellent example of how not to draw unwarranted conclusions based on an insufficient number of tests with very large standard deviations. The statistical method of analysis presents a logical interpretation of a large number of control tests. However, it is hoped that the author is not entirely satisfied with the practice of the U. S. Bureau of Reclamation on compacting earth dams.

The compaction of 50,000,000 cubic yards of various soils by "essentially the same type and amount of compactive effort" is certainly wasteful. As listed in Table 1, the materials for the 39 dams that had been compacted vary from sand and gravel with fines to plastic clays. And the laboratory minus No. 4 densities vary from 100.4 to 132.2 pounds per cubic foot, with the optimum moisture contents from 8.9 to 21.4 per cent. It is reasonable to expect that the compactive effort necessary to obtain the required densities varies greatly for such a large variety of soils with wide range of moisture contents. For instance, it may appear that the material for dams such as Jackson Gulch Zone 1, North Coulee, Jamestown, Vermejo 2, 3, and Stubble could all likely be compacted to satisfactory densities with a lighter roller and less number of passes than had been used. It is oftentimes of practical necessity for small earth dams employing limited design and supervising personnel to generalize specifications such as the Bureau's 12 passes and 6-inch compacted layer with a heavy tamping roller. But the possible savings in time and money with specifications designed for the particular material to be used on a large earth dam should be seriously considered.

It will be interesting to know the experimental evidence of the Bureau's current specification for tamping rollers which would rule out some of the commercial rollers. The Corps of Engineers experience on soils for Blakely Mountain Dam[1] was that the tamping roller with foot size of 14 square inches

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* Resident Engr., Quebradona Dam, Medellin, Colombia; Gannett, Fleming, Corddry and Carpenter, Inc., Harrisburg, Pa.

1. "Review of Soil Design Construction and Prototype Analysis Blakely Mountain Dam, Arkansas," Technical Report No. 3-439, Waterways Experiment Station, Vicksburg, Miss., Oct. 1956.

gives a better result than that with 7 square inches, and the roller weight used for a satisfactory result was only 2,800 pounds per foot of drum. The writer's experience for material with high moisture content at Quebradona Dam, Colombia, was that the tamping foot area as high as 21 square inches gave the best result. It may seem, therefore, that the Bureau's specification of maximum foot size of 10 square inches gives the best result only to certain types of soil. Also, the specification with heavy tamping roller capable of exerting 4,000 pounds per foot is not considered necessary when the field moisture content is relatively high because the compactive effort required for compacting a soil decreases rapidly with its moisture content.

It is true that adequate compaction can be obtained in a variety of soils whether the rollers walk out or not. For the tamping roller which does not walk out at all, the compacting is being done to the layer underneath and under a reduced pressure due to the roller drum contacting the soil and the frictional resistance on the tamping feet by the surface soil layer. More compactive energy is therefore needed to compact consistently the underneath layer than the surface layer to obtain the same density. The soil may be compacted to the same density with a less leavy roller which would result in walk out or partial walk out.

In the conclusion the author states: "Bureau experience indicates that design criteria based on result of laboratory tests have been essentially fulfilled or exceeded in the field." The interesting question to ask after reading this statement would be: "At what cost?" It is felt that modern earth dams should be designed for a density which is not necessarily the practical maximum but the most economical. The cost of soil compaction and the improved means for reduction of cost of compaction should be one of the principle concerns of earth dam engineers. It is hoped that more information can be obtained from the analysis of experimental data on the comparative cost basis and for a better understanding of the fundamental relationships of the various factors in the compacting process.

DEWATERING MIAMI'S BISCAYNE AQUIFER[a]

Discussion by W. Watters Pagon

W. WATTERS PAGON, M. ASCE.—The author has presented an excellent and informative discussion of a difficult unwatering undertaking. He has also given a thorough outline of the local geologic formations; and it is for this that the writer offers some further data.

The writer was fortunate in having studied under Dr. George B. Shattuck, who was the exponent of the "terrace formations," under direction of Dr. Wm. Bullock Clark, State Geologist of Maryland. These are outlined on Page 9.

During the course of many years the writer has accumulated an extensive record of boring logs from which the following is drawn. These terraces are now sub-aerial; he has identified (to his own satisfaction) that there are also three (or perhaps more), to be found at depths of -30', -60' and -90' or 100' The logs in question deal with the two shores of the Patapsco R., from Baltimore Harbor to its juncture with the Chesapeake Bay; the greatest number were located on the north shore where the terrain is markedly different from the south shore; but there is one set, with the greatest number in one cluster, on the south shore where the Harbor Tunnel (not under construction) passes under a new bridge.

The north shore area is Talbot Terrace, with Wicomico and higher ones further inland. Correlation of the logs has been relatively easy because some logs disclose two evident deposit levels. At one spot there is a large structure which spans the escarpment between the two lower lying ones, and a spot in the open water of the River showing the same pairing. In Bear Creek, a tributary, the writer had occasion to reinforce a long double track pile trestle with lengths specified from five logs, and of a large number ordered 85' to 90' only five or six failed to meet refusal in the underlying Patuxent sand-gravel formation. At a point nearby, with about +5' to 0' of silty-clay and 0' to -100' or more of "mud" with occasional shells which showed some evidence of slight consolidation (as though exposed at some time, which seems improbable).

Of greater interest is that mentioned above. The first few logs showed an apparently erratic condition, hence there were made three or more per bent. When plotted a diagonal line at nearly 45° gave evidence of the foreshore, the "break-point" of the waves, and the escarpment. * * * There is some evidence (perhaps intuitive) in the logs for the Chesapeake Bay Bridge, in the deeper water of the Bay, and in the lower Cretaceous formations, that there may be other, deeper terraces.

The author's title "Depth water formed in" is somewhat misleading, since each such terrace was at zero water level of the time. Naming, for convenience in order Baltimore, Arundel and Patapsco, and reversing those listed on Page 9, we would have; 30, 60, 100, 142, 170, 200, 270, 315, 370, (and perhaps others). * * * * It would be of interest if there can be found similar traces in Florida.

a. Proc. Paper 1299, July, 1957, by Byron J. Prugh.

DETERMINATION OF THE 0.02 MM. FRACTION IN GRANULAR SOILS[a]

Discussion by H. Y. Fang

H. Y. FANG,* A.M. ASCE.—The determination of soil particles smaller than No. 200 sieve by the hydrometer test is one of the most time-consuming tests associated with routine soil analysis. Mr. Johnson is to be commended for his statistical approach to this investigation, since this approach will greatly reduce the time required to obtain pertinent information regarding soil grain size. However, the factor determined by Mr. Johnson's statistical analysis is not a constant for all materials. This constant depends upon the properties of the material passing the No. 200 sieve. It will be necessary to establish this constant by making a thorough study of local material, or the soil type under consideration.

In determining the factor, Mr. Johnson has used the arithmetic mean, median, and mode. These methods are well recognized for obtaining averages by simple computation. However, the arithmetic mean may be greatly distorted by extreme values. The median has larger standard and probable errors than the arithmetic mean. When the number of values is limited, the mode significance is also limited. The best determination of such a constant is through the use of the lease-square method. i.e., through minimization of the sum of the squares of the deviations from the observed. The sum of squared deviations between observed and the calculated values will then be a minimum. It can be expressed by formula as:

$$\sum_{i=1}^{i=n} e^2 = \text{minimum}$$

Let:

Y_1 = % passing No. 200 sieve.
Y_2 = % passing 0.02 mm.
Y_3 = % passing 0.005 mm.
Y_4 = % passing 0.002 mm.

The general equations are:

$$Y_2 = a_2Y_1 + b_2 + e_2 \tag{1}$$

$$Y_3 = a_3Y_1 + b_3 + e_3 \tag{2}$$

$$Y_4 = a_4Y_1 + b_4 + e_4 \tag{3}$$

Where a_2, a_3, and a_4, b_2, b_3 and b_4 are constants to be determined.

e_2, e_3 and e_4 are individual errors, or deviations between observed and calculated values for Y_1, Y_2, Y_3 and Y_4.

Equation (1) may be written as:

$$Y_{2i} = a_2Y_{1i} + b_2 + e_{2i}$$

Where i = 1, 2, 3, -------n

a. Proc. Paper 1309, July, 1957, by R. W. Johnson.

* Soils Eng., AASHO Road Test, National Academy of Sciences, Ottawa, Ill.

By the least square criterion:

$$\sum_{i=1}^{i=n} e_{2i}^2 = \text{minimum}$$

or

$$\sum_{i=1}^{i=n} (Y_{2i} - a_2Y_{1i} - b_2)^2 = \text{minimum}$$

We may then obtain the partial derivatives with respect to a_2 and b_2, and equate to zero to obtain a value for a_2 and b_2 which will make $\Sigma\ e_{2i}^2$ a minimum.

$$\frac{\partial \sum_{i=1}^{i=n} e_{2i}^2}{\partial a_2} = 2\sum_{i=1}^{i=n} (Y_{2i} - a_2Y_{1i} - b_2)(-Y_{1i}) = 0$$

$$\frac{\partial \sum_{i=1}^{i=n} e_{2i}^2}{\partial b_2} = 2\sum_{i=1}^{i=n} (Y_{2i} - a_2Y_{1i} - b_2) = 0$$

or

$$a_2 \sum_{i=1}^{i=n} Y_{1i}^2 - b_2 \sum_{i=1}^{i=n} Y_{1i} = \sum_{i=1}^{i=n} Y_{2i}Y_{1i}$$

(A)

$$a_2 \sum_{i=1}^{i=n} Y_{1i} - Nb_2 = \sum_{i=1}^{i=n} Y_{2i}$$

Derivation of a_3, b_3, a_4 and b_4 are the same as a_2 and b_2. Therefore:

$$a_3 \sum_{i=1}^{i=n} Y_{1i}^2 - b_3 \sum_{i=1}^{i=n} Y_{1i} = \sum_{i=1}^{i=n} Y_{3i}Y_{1i}$$

(B)

$$a_3 \sum_{i=1}^{i=n} Y_{1i} - Nb_3 = \sum_{i=1}^{i=n} Y_{3i}$$

$$a_4 \sum_{i=1}^{i=n} Y_{1i}^2 - b_4 \sum_{i=1}^{i=n} Y_{1i} = \sum_{i=1}^{i=n} Y_{4i}Y_{1i}$$

(C)

$$a_4 \sum_{i=1}^{i=n} Y_{1i} - Nb_4 = \sum_{i=1}^{i=n} Y_{4i}$$

Forty varied soil samples were tested in our laboratory. The physical properties of these soils are as follows:

Liquid Limit = 22 - 36
Plasticity Index = 9 - 18
Specific Gravity = 2.69 - 2.76

SUMMARY SHEET ON SIEVE AND HYDROMETER ANALYSIS

Table No. 1

Test No.	(% passing)							
	SIEVE ANALYSIS					HYDROMETER ANALYSIS		
	#4	#10	#40	#60	#200	0.02mm.	0.005mm.	0.002mm
1	98.3	94.7	89.3	86.5	78.5	56.0	33.0	20.0
2	99.0	95.3	90.0	87.4	79.1	59.0	36.0	21.0
3	98.3	94.5	89.5	87.1	79.0	60.0	37.0	21.0
4	98.4	95.0	89.3	86.3	78.1	60.0	36.0	24.0
5	98.0	94.8	89.7	87.3	78.0	63.0	42.0	26.0
6	99.0	95.7	89.8	87.2	79.0	64.0	43.0	28.0
7	98.6	94.8	89.2	86.8	79.0	63.0	40.0	26.0
8	98.2	94.7	89.3	86.8	80.0	63.0	40.0	24.0
9	99.0	95.8	90.7	88.6	82.7	65.4	44.2	27.1
10	98.9	96.0	91.8	88.5	81.0	65.8	47.8	28.0
11	97.9	94.1	89.4	87.3	78.0	64.0	46.0	30.0
12	99.1	95.8	90.8	88.7	82.8	68.0	46.0	30.0
13	97.5	93.2	88.7	86.5	79.1	58.0	37.0	20.0
14	98.8	95.7	89.8	86.7	77.8	57.0	36.0	21.0
15	98.3	94.7	92.0	87.5	79.6	59.0	33.0	23.0
16	98.5	95.2	89.5	86.5	78.1	61.0	40.0	24.0
17	99.2	96.2	90.6	87.6	78.4	57.0	36.0	23.0
18	98.4	94.3	88.7	85.9	76.4	57.0	37.0	18.0
19	98.2	96.2	90.4	87.3	76.4	59.0	35.0	22.0
20	98.5	97.0	90.2	87.1	76.0	58.0	34.0	18.5
21	99.2	95.7	89.8	86.9	78.0	57.0	37.0	21.0
22	98.9	97.9	93.7	91.9	82.0	67.0	46.0	23.0
23	99.0	97.5	93.0	91.1	85.5	69.0	51.0	29.0
24	99.0	97.8	94.2	92.5	86.9	70.0	46.0	26.0
25	98.3	97.5	92.1	89.8	81.7	66.0	46.0	20.0
26	98.7	96.2	90.4	87.9	80.5	67.0	47.0	25.0
27	97.4	94.8	90.3	88.3	82.0	63.0	42.0	28.0
28	97.9	96.9	91.5	84.5	80.4	60.0	37.0	25.0
29	99.0	97.6	91.2	83.7	78.1	60.0	38.0	18.0
30	99.2	97.1	91.5	89.0	81.0	66.0	46.0	25.0
31	98.7	95.4	90.4	86.8	76.4	55.0	34.0	19.5
32	99.6	96.2	90.2	87.4	80.2	57.5	35.5	20.0
33	97.6	95.4	89.6	88.2	81.0	57.5	35.5	21.0
34	98.6	95.4	89.2	89.4	78.3	56.5	39.0	22.0
35	99.1	96.2	90.1	86.4	79.6	57.5	40.0	25.0
36	99.0	96.4	90.3	86.7	79.4	58.0	38.0	24.0
37	99.4	96.0	90.0	85.9	80.0	58.0	39.0	25.0
38	89.4	82.0	70.3	64.0	54.0	38.0	24.0	16.0
39	97.4	94.3	90.1	85.8	76.1	58.0	34.5	21.5
40	86.3	81.4	71.0	66.0	52.5	36.0	18.0	10.0

The standard AASHO test procedure (AASHO T88-42) was followed in the laboratory along with routine calculations. Grain size curves were drawn for each soil.

Table I shows the combined results of sieve and hydrometer analysis. From this table we may compute all constants by using the above equations. In order to solve for a_2, a_3, a_4, b_2, b_3 and b_4 the following values are necessary:

$$N = 40$$

$$\Sigma Y_1^2 = 246546.34$$

$$\Sigma Y_1 = 3130.6$$

$$\Sigma Y_1Y_2 = 188861.93$$

$$\Sigma Y_2 = 2394.2$$

$$\Sigma Y_3Y_1 = 122746.89$$

$$\Sigma Y_3 = 1552.5$$

$$\Sigma Y_4Y_1 = 72582.22$$

$$\Sigma Y_4 = 918.6$$

Substitute in equations (A) (B) and (C). We have:

$$\left.\begin{aligned} 246546.34\,(a_2) - 3130.6\,(b_2) &= 188861.93 \\ 3130.6\,(a_2) - 40\,(b_2) &= 2394.2 \end{aligned}\right\} \quad (A)'$$

$$\left.\begin{aligned} 246546.34\,(a_3) - 3130.6\,(b_3) &= 122746.89 \\ 3130.6\,(a_3) - 40\,(b_3) &= 1552.5 \end{aligned}\right\} \quad (B)'$$

$$\left.\begin{aligned} 246546.34\,(a_4) - 3130.6\,(b_4) &= 72582.22 \\ 3130.6\,(a_4) - 40\,(b_4) &= 918.6 \end{aligned}\right\} \quad (C)'$$

The equations may be solved simultaneously.
In equation (A)' we obtained:

$$a_2 = 0.967$$

$$b_2 = -15.8$$

In equation (B)' we obtained:

$$a_3 = 0.811$$

$$b_3 = -24.7$$

In equation (C)' we obtained:

$$a_4 = 0.450$$

$$b_4 = -12.3$$

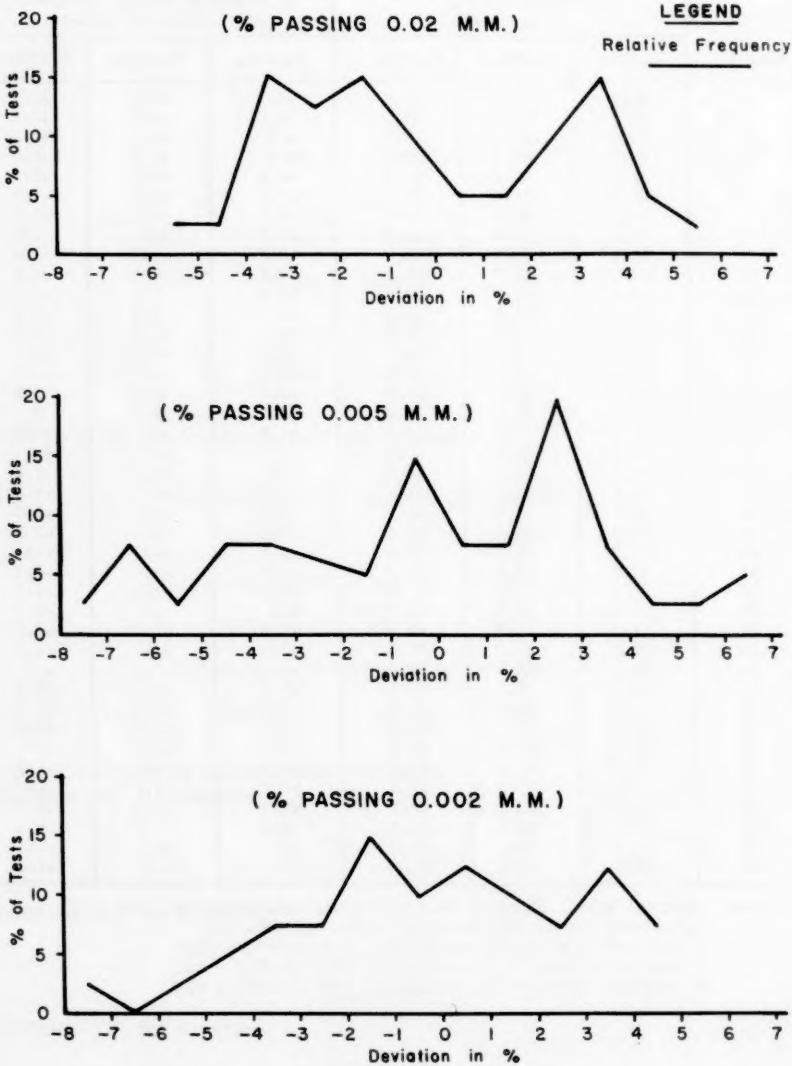
COMPARISON BETWEEN OBSERVED
AND CALCULATED VALUES
Table No. 2

Test No.	Calculated values % passing			Deviations (Difference between observed and calculated values)		
	0.02mm	0.005mm	0.002mm	$d_{0.02mm}$	$d_{0.005mm}$	$d_{0.002mm}$
1	60.1	39.0	23.0	+4.1	+6.0	+3.0
2	60.7	39.5	23.3	+1.7	+3.5	+2.3
3	60.6	39.4	23.3	+.6	+2.4	+2.3
4	59.7	38.6	22.8	-.3	+2.6	-1.2
5	59.6	38.6	22.8	-3.4	-3.4	-3.2
6	60.6	39.4	23.3	-3.4	-3.6	-4.7
7	60.6	39.4	23.3	-2.4	-.6	-2.7
8	61.6	40.2	23.7	-2.4	+.2	-.3
9	64.2	42.4	24.9	-1.2	-1.8	-2.2
10	62.5	41.0	24.2	-3.3	-6.8	-3.8
11	59.6	38.6	22.8	-4.4	-7.4	-7.2
12	64.3	42.5	25.0	-3.7	-3.5	-5.0
13	60.7	39.5	23.3	+2.7	+2.5	+3.3
14	59.4	38.4	22.7	+2.4	+2.4	+1.7
15	61.2	39.9	23.5	+2.2	+6.6	+.5
16	59.7	38.6	22.8	-1.3	-1.4	-1.2
17	60.0	38.9	23.0	+3.0	+2.9	0
18	58.1	37.3	22.1	+1.1	+.3	+4.1
19	58.1	37.3	22.1	-.9	+2.3	+.1
20	57.7	36.9	21.9	-.3	+2.9	+3.4
21	59.6	38.6	22.8	+2.6	+1.6	+1.8
22	63.5	41.8	24.6	-3.5	-4.2	+1.6
23	66.9	44.6	26.2	-2.1	-6.4	-2.8
24	68.2	45.8	26.8	-1.8	-.2	+.8
25	63.2	41.6	24.5	-2.8	-4.4	+4.5
26	62.0	40.6	23.9	-5.0	-6.4	-1.1
27	63.5	41.8	24.6	+.5	-.2	-3.4
28	61.9	40.5	23.9	+1.9	+3.5	-1.1
29	59.7	38.6	22.8	-2.3	+.6	+4.8
30	62.5	41.0	24.2	-3.5	-5.0	-.8
31	58.1	37.3	22.1	+3.1	+3.3	+2.6
32	61.8	40.3	23.8	+4.3	+4.8	+3.8
33	62.5	41.0	24.2	+5.0	+5.5	+3.2
34	59.9	38.8	22.9	+3.4	-.2	+.9
35	61.2	39.9	23.5	+3.7	-.1	-1.5
36	61.0	39.7	23.4	+3.0	+1.7	-.6
37	61.6	40.2	23.7	+3.6	+1.2	-1.3
38	36.4	19.1	12.0	-1.6	-4.9	-4.0
39	57.8	37.0	21.9	-.2	+2.5	+.4
40	35.0	17.9	11.3	-1.0	-.1	+1.3

Notes: Observed values obtained from laboratory hydrometer analysis
Calculated values obtained from following formulas:
% passing 0.02mm. = (% passing No. 200) (0.967) - 15.8
% passing 0.005mm. = (% passing No. 200) (0.811) - 24.7
% passing 0.002mm. = (% passing No. 200) (0.450) - 12.3

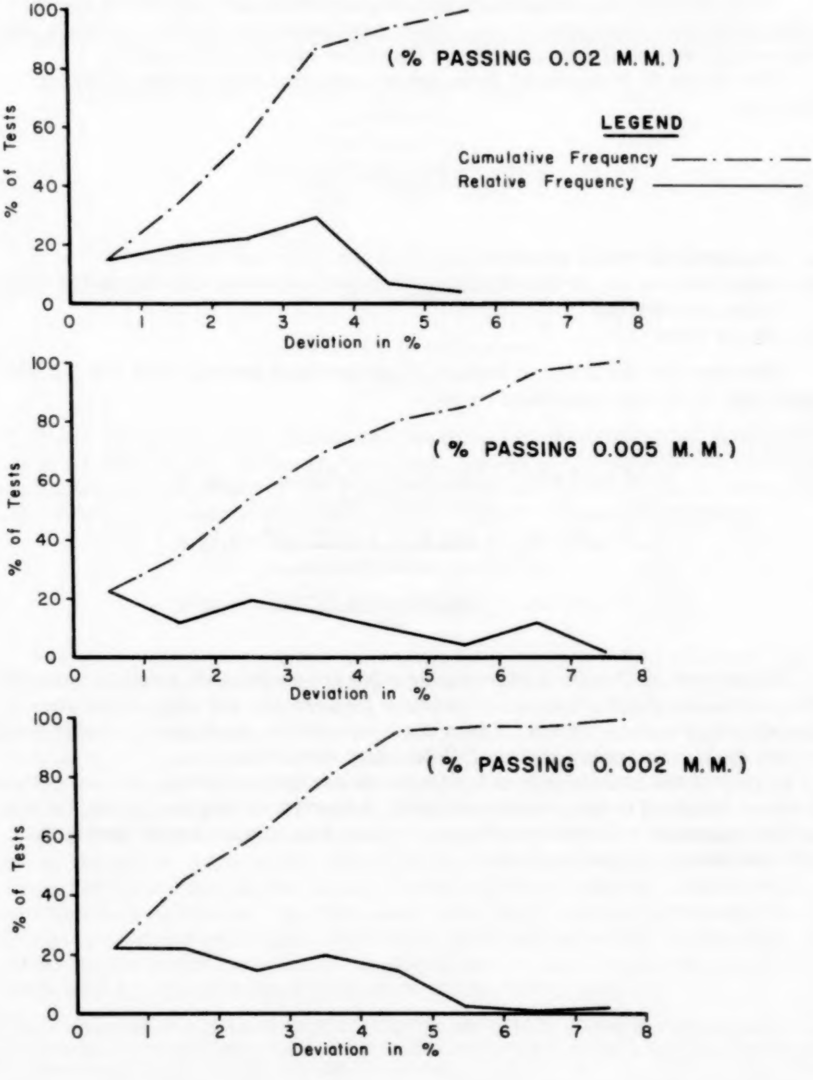
FREQUENCY POLYGONS of ALGEBRAIC ERRORS

Fig. No. I



FREQUENCY POLYGONS of ABSOLUTE ERRORS

Fig. No. 2



From the above statistical analysis, the following relations may be established, and estimates of "percentage material passing" can be computed.

% passing 0.02 mm. = (% passing No. 200) (0.967) - 15.8
% passing 0.005 mm. = (% passing No. 200) (0.811) - 24.7
% passing 0.002 mm. = (% passing No. 200) (0.450) - 12.3

Table 2 shows the comparison between observed and calculated values. The observed values are obtained from laboratory hydrometer analysis, and calculated values are obtained from the above statistical formulas.

The standard deviation of these errors was computed by the following formula.

$$\sigma = \sqrt{\frac{\Sigma d^2 - (\Sigma d)^2/N}{N-1}}$$

σ = Standard deviation of error
d = Individual error, or the deviations between observed and calculated values. (sign considered)
N = No. of tests

The standard deviation of errors in percentages passing 0.02 mm., 0.005 mm. and 0.002 mm. are found to be

$$\sigma\ 0.02\ \text{mm.} = \sqrt{\frac{315.67 - (-1.9)^2/40^*}{40 - 1}} = 2.84\ \%$$

$$\sigma\ 0.005\ \text{mm.} = \sqrt{\frac{493.67 - (-1.3)^2/40^*}{40 - 1}} = 3.55\ \%$$

$$\sigma\ 0.002\ \text{mm.} = \sqrt{\frac{327.29 - (-1.7)^2/40^*}{40 - 1}} = 2.90\ \%$$

The curves on Figure 1 show the relative frequencies of algebraic errors. The curves on Figure 2 show the relative frequencies and cumulative frequencies of absolute errors. From these curves the maximum probable error of this study was approximately $\pm$ 2 standard deviations.

In view of the time required for thorough laboratory testing and the personal error involved in the results obtained, it is evident that the use of this statistical approach will reduce the time required to obtain results consistent with laboratory testing methods.

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* Note: Σ d, theoretically will be equal to zero, but numerical values shown here are obtained by rounding off the decimals.

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GARRISON DAM-TUNNEL TEST SECTION INVESTIGATION - A SYMPOSIUM

Harris H. Burke,[1] M. ASCE
(Proc. Paper 1438)

Evaluation of Test Results and Application to Tunnel Design

K. S. Lane

ABSTRACT

These two papers describe an extensive 6-year investigation of tunnel loading in a large 36-foot diameter tunnel test section at Garrison Dam; including application of the results to the 8 prototype tunnels, construction of these tunnels through clay-shale, and observations on their initial performance.

FOREWORD

Design of temporary support and permanent lining for the tunnels at Garrison Dam was approached by early construction of a 240-foot length in one of the 8 main tunnels and equipping this with a variety of measuring facilities to serve as a test section. This procedure was chosen because of the absence of tunnelling experience in clay-shale and the tunnels' relatively large size—from 27 to 36-foot diameter. Measurements were continued in several of the prototype tunnels with a total observation period of about 6 years.

Resulting cost savings in the main tunnels were very substantial and many times the not inconsiderable expense of the investigation. Furthermore, the investigation placed the design on a firmer basis and afforded considerable assurance of satisfactory performance in service. Although problems of tunnel loading are so complex that close solutions should not be expected, the investigation added considerable to the limited present knowledge and should be helpful for future tunnels in weak rocks or strong soils.

Note: Discussion open until April 1, 1958. Paper 1438 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, Vol. 83, No. SM 4, November, 1957.

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Noteworthy contributions to design concepts were a clearer insight into the action of a circular lining in carrying the tunnel load and striking evidence of the influence of lining stiffness on this load. Construction innovations included a method for nearly continuous mucker operation and special jumbos with well planned safety features which contributed to the remarkable record of 10,000 feet of large tunnel without a single fatality.

The first paper by Mr. Burke describes the conduct of the tunnel investigation, its principal results and the construction of the main tunnels.

The second paper by Mr. Lane deals with the evaluation and application of the results and includes the development of some new theoretical concepts.

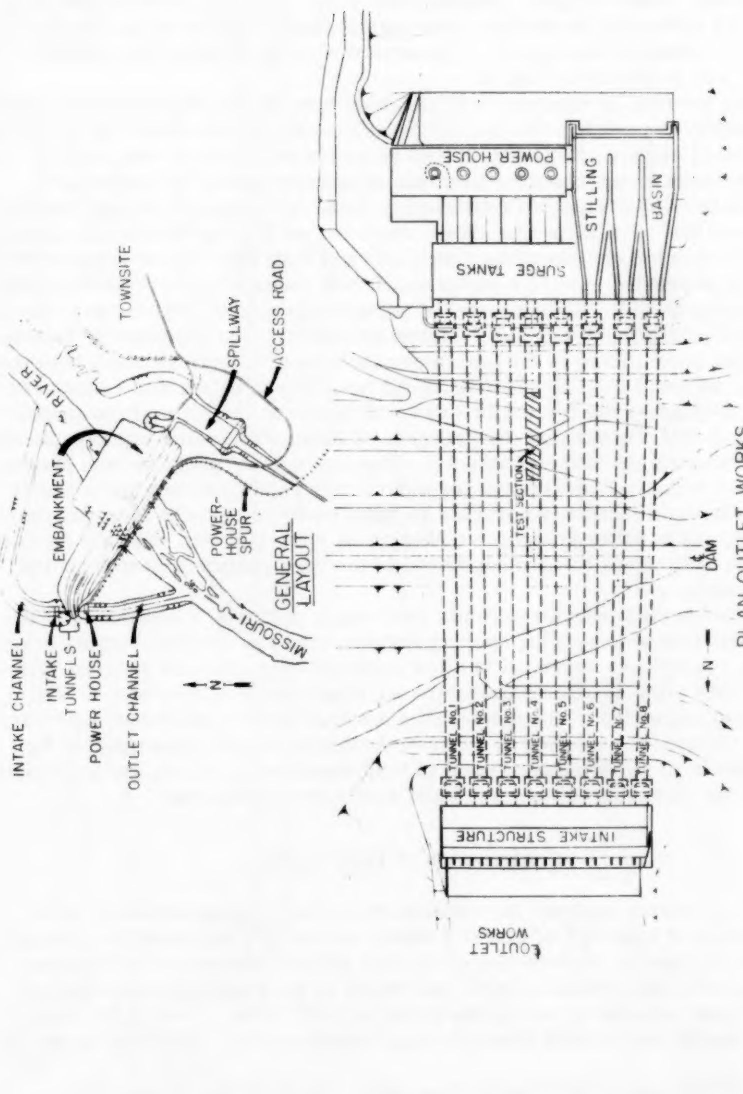
SYNOPSIS

One of the major design problems encountered on the Garrison Dam Project was determination of magnitude and distribution of earth pressure on the eight nearly parallel tunnels which form part of the outlet works. Originally, it was planned to utilize cut and cover conduits located on the flood plain west of the river for the outlet works but difficult foundation conditions led to the decision to use multiple tunnels. In order to check the basic assumptions used in the design of the tunnels an extensive program of measurements and observations on a full scale section of one of the tunnels was undertaken. This paper describes the test section investigation, the design and construction of the main tunnels, and presents the results of observations both in the test section and at selected observation stations throughout six of the eight tunnels. A companion paper by Mr. K. S. Lane discusses the evaluation and application of the observation results to the design of the Garrison tunnels and the possible extension of these results to the design of other tunnels. Only typical results and the more significant studies are presented; the full investigation is covered by Corps of Engineers reports[2] filed in the ASCE Library and in the library of the U.S. Waterways Experiment Station, Vicksburg, Miss.

INTRODUCTION

The now nearly complete multiple purpose Garrison Dam is one of the major units of the Pick-Sloan Plan, the comprehensive plan for the control of the Missouri River system. Garrison Dam, located about 75 miles north of Bismarck, North Dakota, is rolled fill embankment with a maximum height of more than 200 feet and a length greater than two miles. A description of the project constructed by the Corps of Engineers has been given by J. S. Seybold.[3] The outlet works, located on the west abutment as shown in Fig. 1 consist of an intake structure, eight circular tunnels about 1200 feet long, power facilities, and a stilling basin. Five of the tunnels are for power and were excavated to a bore of 35 feet and lined with 3 feet of concrete. Two of the regulating tunnels are 22 feet inside with 2-1/2 feet of concrete lining. The last regulating tunnel is 26 feet inside with 2 feet 9 inches of concrete

2. "Report of Test Tunnel, Garrison Dam," Garrison District, Corps of Engineers, Part 1, August 1949; and Part II, July 1953.
3. "Constructors Roll Nearly One Million Yards a Week into Garrison Dam," by J. S. Seybold, Civil Engineering, October 1949.


INTAKE CHANNEL
INTAKE TUNNELS
POWER HOUSE
OUTLET CHANNEL
N
EMBANKMENT
RIVER
MISSOURI
TOWNSITE
SPILLWAY
ACCESS ROAD
POWER-HOUSE SPUR
GENERAL LAYOUT
INTAKE STRUCTURE
℄ OUTLET WORKS
TUNNEL No.1
TUNNEL No.2
TUNNEL No.3
TUNNEL No.4
TUNNEL No.5
TUNNEL No.6
TUNNEL No.7
TUNNEL No.8
TEST SECTION
℄ DAM
SURGE TANKS
POWER HOUSE
STILLING
BASIN
N
PLAN OUTLET WORKS
100'
200'
FIG. 1

lining. The tunnels were constructed with temporary steel rib support which was encased by the permanent lining of reinforced concrete.

The tunnels were mined through the Fort Union formation,[4] a compact clay shale of Tertiary age which consists of continentally deposited gently dipping beds ranging from clays to sands. The beds are often cross-bedded, and range from thin partings to 15 or more feet thick with the clayey phases predominating. Beds of lignite ranging from a few inches to several feet thick are found within the formation. Two major lignite beds were encountered within the plane of the tunnels. The locations of the tunnels are shown in section and in profile in Fig. 2.

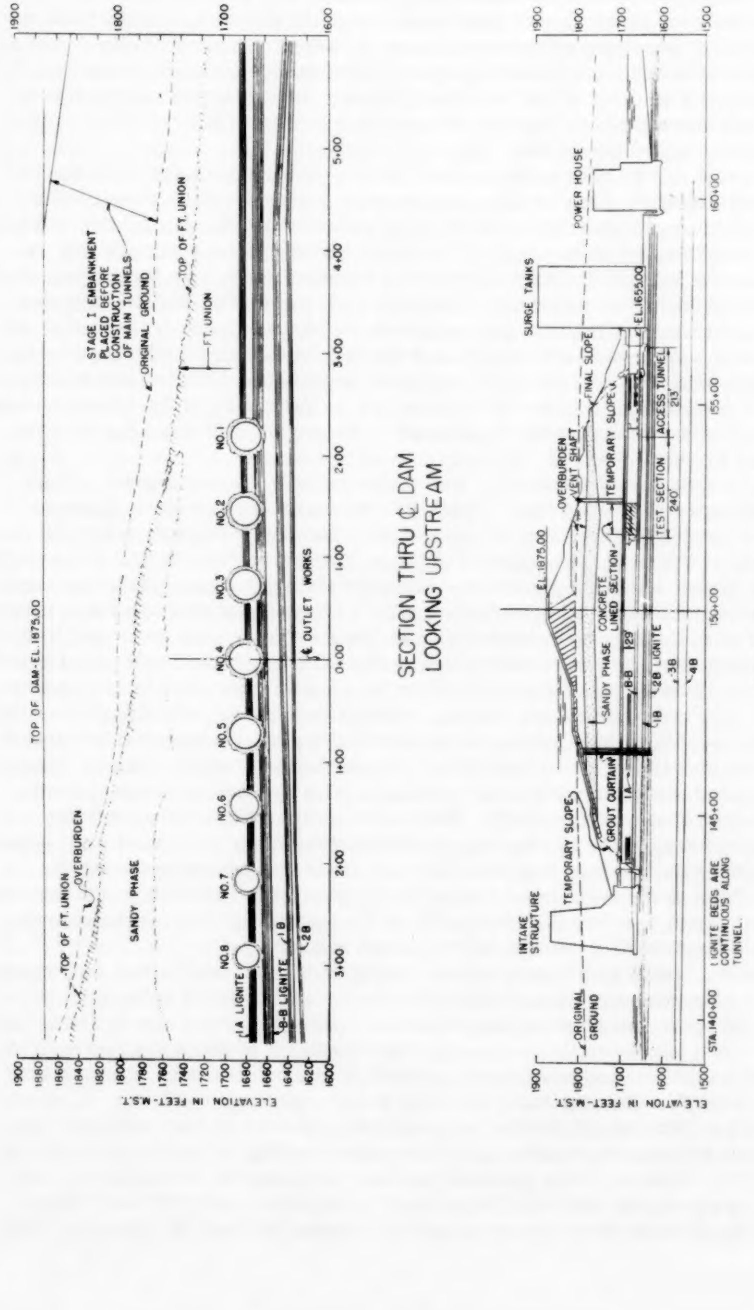
In the absence of experience in tunneling through the Fort Union formation, the board of Consultants for the Garrison Project recommended early in 1947 that a short section of full size tunnel be mined as a test section, or test tunnel, and that a comprehensive program of measurements be instituted in order to develop design and construction data. In the tunnel design studies, it developed that very little was known about mining through formations similar to the Fort Union and about the magnitude and distribution of pressures which would be developed in such a material. It was thought the Fort Union might be an energetically swelling material when the preconsolidation due to around 1000 feet of overburden, since removed by erosion, was released by mining the tunnel bore. Some of the laboratory tests on drill hole samples indicated swelling properties while other tests did not. The problem was considered to be complicated by the two major seams of lignite at the level of the tunnels although it was thought that the presence of these coal seams would probably be favorable rather than detrimental. The test section program was planned to furnish technical information for development and confirmation of the design of the main tunnels, to provide an opportunity to develop construction methods, and to allow prospective bidders an opportunity to evaluate the construction difficulties to be encountered in the construction contract for the main tunnels.

The test section was divided into four major portions, a section of 12 segment steel ribs with varying spacing between ribs and between lagging in each of three subsections arranged to allow yield between ribs and between lags, a section with wood crusher blocks between segments to allow yield of ribs under load, a section of fixed steel ribs similar to the main tunnel construction, and a section with slotted concrete lining having the slots spanned by heavy beams to allow measurement of load reactions. Transitions were provided at the ends and between the steel and concrete portions.

Construction of Test Section

The first major contract for excavation and main embankment included construction of a section of tunnel 4 which served as a full scale test tunnel. In order to expedite construction of the test section, an access cut totaling approximately 890,000 cubic yards was made in the Stage I powerhouse and stilling basin excavation during the winter of 1947-1948. Most of the excavated material was wasted although major lignite layers, three feet or more

4. "Soil Properties of Fort Union Clay Shale," by C. K. Smith and J. F. Redlinger, Proc. Third International Conference on Soil Mechanics and Foundation Engineering, Switzerland, 1953.



SECTION THRU ℄ DAM LOOKING UPSTREAM

PROFILE - TUNNEL NO.4

FIG 2

thick, were salvaged and stockpiled. The test section, including an access tunnel, was constructed during the period from 29 March 1948 to 26 February 1949. The behavior of the Fort Union formation, through which the tunnel was mined, favored rather than retarded progress. The construction rate of the test tunnel was much slower than was anticipated due to operational and organizational problems of the contractor. However, the slower rate of progress allowed sufficient time to make more comprehensive measurements than had been planned on most of the test installations. Literally thousands of measurements were made during the construction period with little or no conflict with construction operations.

To reach the full bore test section, located far enough back from the temporary excavation slope to take a substantial load, an access tunnel 9 feet wide and 11 feet high was mined by hand methods into the Fort Union 213 feet from the portal. The face was drilled in a varying pattern, dictated by experience, to a depth of about 6 feet using six point, side hole jackhammer bits and later 2 inch coal auger bits turned by electric drills. Electrically detonated permissible dynamite was used to break out the face. The blasted material was hand loaded into dump cars which traveled on 36 inch gage track and were hauled in and out of the tunnel by a cable and air powered winch. About 2 feet of lignite coal, left in place above the crown of the access tunnel, formed a competent roof which allowed timbering to lag behind the face as much as 60 feet.

The access tunnel excavation was continued into the test section proper for a distance of about 6 feet. From this extension a shaft about 6 feet by 7 feet was raised to the crown of the test section. The shaft was enlarged laterally at the top of the upper, 1-A, lignite to approximately full tunnel size and the crown above the lignite was temporarily supported by a timber arch framework. The lateral enlargement was continued downward to form a short section of full circle tunnel and the first two steel ribs were erected. The top portion of the full bore enlargement was then lengthened both forward and backward. During this enlargement the crown was supported on temporary timber ribs with tight timber lagging. Water seeping into the tunnel from the 1-A lignite greatly hindered operations in making the raise and enlargement. While the quantity of water was small, it complicated construction by causing swelling and sloughing of the clay and especially by causing slippery footing in the restricted working space. To eliminate this undesirable condition mining was suspended and the lignite was grouted from horizontal drill holes. Subsequent grouting was resorted to at intervals as necessary during the construction of the test tunnel and generally was quite effective in minimizing seepage. After the initial grouting the slot enlargement was extended to a length of about 20 feet and the initial jumbo was erected.

After the jumbo had been erected, mining proceeded with a full face operation. The heading was drilled with hand-held electric drills rotating coal augers by operators working both from the jumbo platforms and on the tunnel invert. Both lignite and clay were drilled to a depth of about six feet so that two ribs could be erected each around without an excessive amount of hand excavation. For blasting about ten delays were used and the lower portion of the face was blasted out into the excavated tunnel first so the balance of the face could drop down in successive sections according to the delays used. A Conway No. 90 electrically powered mucker was used for loading muck cars which were pulled in and out of the tunnel by a cable operated from a hoist outside the portal. Final cleanup from each round was accomplished by hand

loading into the mucker bucket. During the mucking operation the crown was supported by 6 inch "I" beams about 12 feet long supported by the rib steel and cantilevered out from the supported section. These beams were moved ahead after each blast. After mucking was completed two 12 segment steel ribs were erected and initial blocking was placed. In general, only enough blocking was placed initially to hold the ribs in proper shape and position through the next blast after which blocking was finished while the blasted material from the next round was being excavated.

In late August 1948 after the test tunnel had been under construction about four months a major change in supervision and work methods was made. A new, more powerful mucker was used in combination with larger muck cars and a battery powered electric locomotive all operating on 24 inch gage track. The jumbo was drastically modified to provide a wider working area for the mucker by placing the supporting wheels on rails near the springline of the tunnel. Excavation and rib erection proceeded essentially as before except that mucking was speeded up considerably by provisions to allow empty and full cars to pass. The test tunnel was mined and ribs set in the six months period from May 3 to November 3, 1948. Mining of the initial slot, enlargement of the bore to allow the jumbo erection, and erection of the jumbo required until July 23; thereafter progress was rather slow but at an increasing rate until August 26 when the construction was reorganized. At that time, about one-fourth of the tunnel had been completed. In the following two months, mining was completed.

The ends of the test section were benched at the levels of the two major lignites and supported by partial circle ribs. The vertical clay faces between the lignites and above the upper lignite had a tendency to slough so transverse bulkheads backfilled with gravel were provided to support the vertical faces. During excavation no significant swelling or squeezing of the Fort Union was noted; it stood in a vertical face permitting full face excavation. The Fort Union behaved as a weak rock breaking out in block-like fragments which ranged up to a cubic yard or more in size. The block-like structure resulted in a varying overbreak which ranged up to three feet in some cases. The average overbreak in the crown half of the test section was 1.8 feet and the average for the entire circular section was 0.7 foot.

Section 4-D at one end of the test section was concreted during the period from 12 November 1948 to 17 January 1949. After placing the invert, which comprised the bottom 60 degrees, both sidewalls were placed simultaneously to the springline followed by a separate pour from the springline to the crown. Concrete was mixed in a 1/2 cubic yard mixer, using natural sand and crushed granite coarse aggregate. The mixer discharged directly into the hopper of a gasoline powered pumpcrete machine which pumped the concrete through an 8-inch line into the invert forms or to an electric powered pumpcrete machine inside the tunnel which relayed the concrete into the side and arch forms. During placing the level in both sides of the side and arch pours was maintained at nearly the same elevation by alternately moving the discharge line from one side to the other. The concrete was internally vibrated by men working inside the forms. End bulkheads were used for the invert and the concrete was screeded to grade. For the side and arch pours steel forms, supplemented by timber end bulkheads, were mounted on a form carriage which was rebuilt after having served as the drill and erection jumbo. A small gallery was excavated above the ribs at the crown to accommodate the concrete pipeline. This gallery was left open after concreting

and was not filled until after the completion of the observation program. After concreting, voids between the concrete and the Fort Union above the springline were grouted with neat cement grout under a low (10 p.s.i.) pressure.

During construction of the test section water seeping from the upper lignite generally was quite effectively stopped by grouting the lignite and cracks in the adjacent clay; however, continued ground movement after completion of the tunnel caused some of the cracks to reopen and allowed water to seep out of the lignites and over the clay. Continued exposure to this slight seepage caused the clay to soften and slough out leaving the lignite intermittently unsupported. The size of the sloughed areas varied considerably with the largest slough about 6 feet deep and 10 feet long. It was recognized early that this sloughing would be detrimental when tunnel 5 was mined alongside but repair of the sloughed areas was delayed until December 1949, when the main tunnel contractor was well established on the project and had men available to accomplish the repair. Then all loose and softened clay was removed and bank-run gravel was tamped into the cavities behind steel lagging which was welded to the tunnel ribs. Where water was actively emerging from the lignite, corrugated sheet metal drains were inserted below the lignite to catch the drainage. In all other areas, minor amounts of seepage emerging from the lignite were allowed to drain down through the gravel and out between the laggings. This gravel filter repair effectively stopped further sloughing and reestablished support of the clay as continued movement tended to tighten the gravel.

Prior to mining tunnel 5 alongside the test section, analysis of observations indicated that crushing of redwood blocks between rib segments in one of two yielding rib sections had not progressed far enough to allow the ribs to develop the strength necessary to prevent excessively high strains in the soil pillar between tunnels 4 and 5 as it took additional load due to mining tunnel 5. The crusher blocks were stressed in their plastic range so that a relatively small increase in load would have resulted in a considerable diameter shortening. The ultimate possible diameter shortening was computed to be about 5 inches in addition to the 2 to 3 inches which had already occurred. Since this appeared to approach the strain limit of the soil and to provide more favorable support for this section under the extreme loading condition expected when mining the adjacent tunnel, the section was strengthened by a concrete lining in January and February 1950. The adjacent smaller yield section, 4-B_2, already had yielded enough that the ultimate possible yield would not overstrain the soil pillar. Accordingly, that section was not reinforced.

Construction of Outlet Tunnels

Description of Design

To achieve overall economy by minimizing the size of the costly intake and outlet structures and the powerhouse as much as possible as well as to minimize the excavations, the tunnels were constructed close together, with a spacing of about one diameter between tunnels. In addition, the tunnels were shortened considerably by cutting steep temporary excavation slopes, generally 1 on 1. Berms were placed in the slopes at lignite layers to catch seepage and the slopes were flattened slightly through a sand stratum near

the ground surface. The temporary slopes were flattened later into permanent slopes by backfill around the portals, Fig. 2. The overall stability of the steep temporary slopes was reduced by mining the tunnels and this required the portal structures to be designed so as to reinforce the slope.[5]

Temporary support of the Fort Union formation during mining was provided by circular steel ribs designed for a maximum working stress of 24,000 p.s.i. To permit use of only one length spacer and lagging all ribs were initially designed for a spacing of 3 feet. The original design of rib steel sizes, given in Table I, was developed in November 1948 when only limited data from the test tunnel were available. Later data from the test section and observations in the main tunnels allowed the spacing to be increased to 4 feet and made possible a reduction in weight of rib steel.

A limited initial supply of steel ribs was supplied by the Government, and included ribs fabricated with both four and eight segments per rib to permit flexibility of operation and thus allow optional mining methods. Later orders were limited to the four segment ribs after experience showed these to be best suited to the adopted erection method. Bolted segment joints were provided with splice plates which were welded to provide tensile strength in the joints equal to a stress of 18,000 p.s.i. on a cross section of steel equal to two-thirds of the rib section.

Reinforced concrete lining 3 feet thick was provided in tunnels 1 through 5 which were 29 feet inside diameter. In tunnel 6, with inside diameter of 26 feet, the concrete lining was 2 feet 9 inches thick and in tunnels 7 and 8, with inside diameter of 22 feet, 2 feet 6 inches of concrete lining was provided. Transverse reinforcing bars, 1-1/4 inches square, were provided on 12 inch centers near the inside face. The temporary support ribs served as the outer transverse reinforcement. Longitudinal reinforcing was provided for stresses caused by temperature, bending due to rebound near the portals and settlement under the embankment, and probable tension in the tunnel barrel due to other factors such as shear stresses from the embankment load, longitudinal rebound near the portals, and consideration of slope stability. A typical cross section of the concrete lining is shown in Fig. 3. In the regulating tunnels where a particularly dense concrete surface was required, the surface of the invert pour was subjected to a vacuum process and absorptive form lining was required for the portion of the side pours below the springline. Pipes placed at regular spacing and at points of high overbreak were provided for grouting voids in the crown.

Mining

As in the test section the Fort Union behaved as a soft rock during mining. Thus, it was possible to complete mining of each tunnel before starting to concrete. Mining was successfully carried forward by blasting and excavating for the full height of the face in rounds of 6 to 8 feet depending on the rib spacing.

Special jumbos of novel arrangement considerably increased progress in mining.[6] The jumbos, mounted on rails just below the springline, cleared

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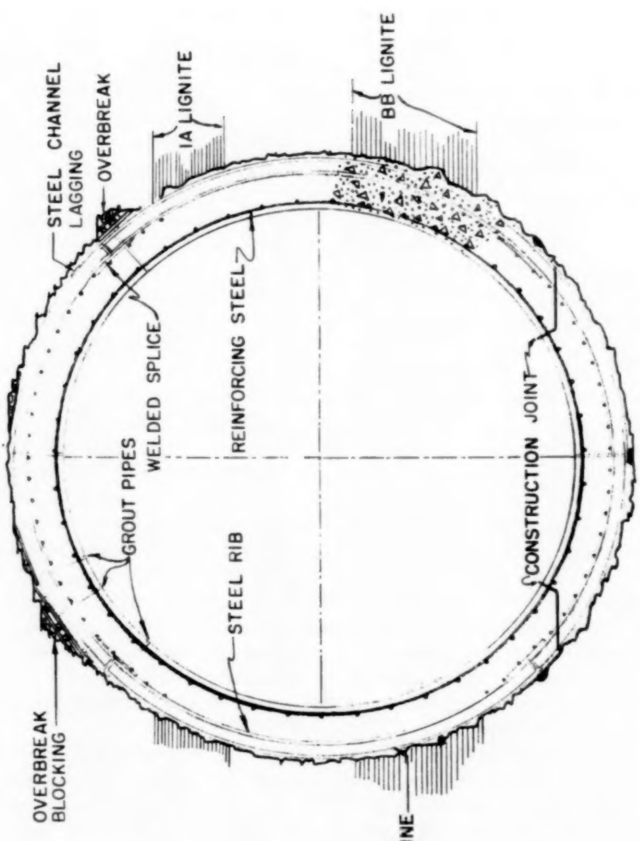
5. "Special Portals for Outlet Tunnels Maintain Stability at Garrison Dam," by K. S. Lane, Civil Engineering, September 1952.
6. "Garrison Tunnels Are Moving Fast," Engineering News Record, January 18, 1951.

SUMMARY OF RIB STEEL SIZES- TABLE I

		Load Condition		Rib Design	
Tunnel No.	Dia. of Tun. (ft)	Max. Over-burden and Surcharge at Center-line of Dam (ft)	Percent of Max. Overburden. (Assumed for design of temp support)	Steel Rib Size for 3 ft. Spacing. (550 to 600 ft. of tunnel length near Centerline of dam where load is a max.)	Steel Rib Size for 3 ft. spacing. (300 to 350 ft. of tunnel length at each end where load is less)
1 - 5	29	145	50	10" WF 72	8" WF 48
6	26	150	40	10" WF 54	8" WF 40
7 & 8	22	160	40	10" WF 49	8" WF 35



TYPICAL CROSS-SECTION

FIG.3

the entire bottom portion of the tunnel thus allowing mucking, setting steel, and drilling to be carried on simultaneously. Two jumbos were provided for the 29 foot diameter power tunnels and one each was provided for the 22 and 26 foot regulating tunnels.

The front, or heading, end of a typical jumbo is shown in Fig. 4. Due largely to the careful design and operation of the jumbos, the eight tunnels were constructed with an excellent safety record; there were no fatal accidents during the main tunnel contract.

After each blast the jumbo was moved up to the face and the crown and upper portion of the face were supported by jacks mounted on the jumbo. Erection of the steel ribs was started with the crown segment and progressed down the sides toward the invert as mucking and drilling for the next round were carried on simultaneously. Drilling started at the crown and progressed toward the invert. By the time the invert was completely mucked and the invert segment of the rib was set, the entire face was drilled and ready for loading for the next blast. Drilling was accomplished with 1-1/2 inch diameter coal augers turned by hand-held air drills. To provide for variations in material the drilling pattern was varied somewhat with a typical loading pattern consisting of 60 to 90 holes loaded with about 500 1/3 pound sticks of permissible dynamite using a primer of 40 percent dynamite for every three sticks of 20 percent dynamite and standard delays from number 0 to number 11. About 0.6 pound of powder was used for each cubic yard of excavation. Figure 5 shows a typical loaded face ready for blasting.

A major factor in the rapid excavation was the electric mucking machines which were modified by addition of a hopper and an air-operated double chute to discharge alternately into cars on a double track behind the mucker.[7] Two air hoists mounted on the rear of the mucker pulled empty cars onto either of two tracks behind the mucker and held the car to the mucker during loading. A similar hoist at the side of the tunnel was used to pull loaded cars back from the mucker. The double discharge chute on the mucker allowed discharge into a car on either track, thus allowing the mucker to operate practically continuously with no time lost for changing cars. Behind the rear limit of travel of the mucker full and empty cars were switched onto the proper track by means of a California type diamond switch. Battery powered locomotives were used for haulage.

Erection of steel ribs proceeded concurrently with the mucking operation, starting with the top segment. This segment was held in place and supported from the jumbo by means of hydraulic and screw jacks. The side segments were raised into position and bolted to the crown segment and, as the last operation after mucking the invert, the invert segment was placed into position and bolted to the side segments.

Wood blocking was used as needed between the bore and the ribs; the specifications required a minimum of 16 blocking points per rib evenly spaced to prevent excessive concentrations of load. About the springline open lagging was used except near the portals for a length of 15 to 20 ribs where solid lagging was used in the top 120 degrees of the crown. Open lagging was spaced with a maximum distance of 3-1/2 feet center to center of lags. Where additional support was considered necessary, the lags were placed closer together. In the top half of the tunnel, 2 inch by 4 inch wire mesh was specified

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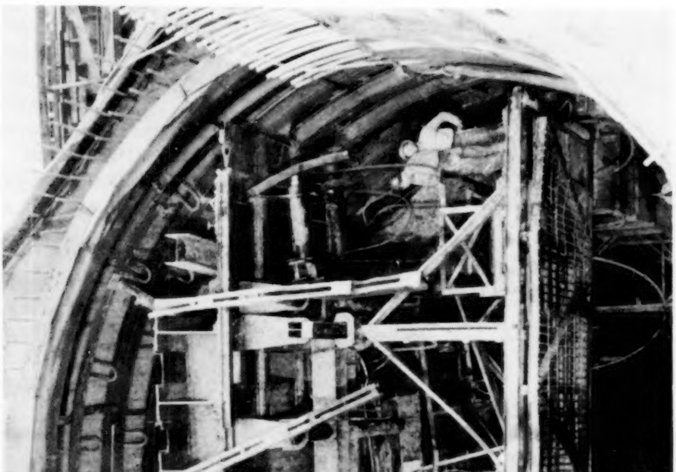
7. Ibid.



FRONT VIEW OF TYPICAL JUMBO
FIGURE 4

LOADED FACE READY FOR BLASTING
FIGURE 5

to catch local falls from the crown. In general, the open lagging served only to support this mesh and ordinarily did not bear against the bore; the load on the ribs was carried entirely through the blocking.

Except for some slaking and local falls, the Fort Union held up extremely well. Due to horizontal bedding planes and vertical joints, the clay had a blocky structure which caused the bore to break in the shape of an inverted stair composed of a series of near vertical and horizontal surfaces. Overbreak was greatest at the crown although throughout all the tunnels overbreak was reasonably small, averaging 0.6 feet outside the outer flangs of the ribs. The largest overbreak encountered during the excavation occurred in tunnel 2 where the Fort Union broke back about 11 feet above the ribs over an area about 12 feet by 15 feet.

Since the lignites which cut through the plans of the tunnels had been exposed in both the intake and powerhouse excavations in early 1948, drainage in the tunnel area had been rather thorough. This considerably reduced the amount of seepage water to be handled during the main tunnel contract and very little water was encountered during mining; however, small amounts of seepage were encountered occasionally in low pockets in the upper, or 1-A, lignite near the springline and in limestone concretions which were found occasionally near the crown. Where seepage spots were encountered, the bore was lagged tight and the space between the lagging and the Fort Union was packed with pit run gravel.

Concreting

Since aggregates suitable for concrete were not available at the site, all aggregates were shipped in by rail. Sand was obtained from a source near Detroit Lakes, Minnesota, with a haul of about 330 miles and coarse aggregates were shipped 205 miles from a source near Greene-Grano, North Dakota. Since no aggregates were shipped during the winter months, stockpiles were built during the summer to allow continuous operation through the winter. Cement was shipped to the site in bottom dump hopper cars and stored in a 7,300 barrel capacity silo.

Aggregates were carried from the stockpiles to bins at the batching plant on a conveyor belt. Cement was transported by an air activated cement conveyor from the storage silo to the batching plant.

The batching-mixing plant located in the powerhouse excavation near the downstream portals had bin capacity for about 500 barrels of cement and sufficient aggregates to produce about 100 cubic yards of concrete. All batching and weighing for the pair of 2 cubic yard tilting mixers was done automatically with tapes recording the weights of each ingredient. Concrete was placed at a maximum temperature of 60 degrees and a minimum temperature of 40 degrees. This narrow range of placing temperatures required cooling the mixing water and the aggregates during the summer and heating aggregates during cold weather.

The 3 inch maximum size aggregate concrete for the portals was transported from the mixing plant to the forms in buckets hauled by trucks. The buckets were handled into the forms by crawler type cranes.

Concrete for the tunnel lining was placed in 24 foot monoliths using 1-1/2 inch maximum size aggregate. Pumpcrete machines were used to transport concrete from the mixing plant into the tunnels. When the pumping distance was greater than could be handled by one pump, booster pumps were

provided at intervals of approximately 800 feet. Concrete for the inverts was pumped directly into place. Where not placed against a previous monolith, bulkhead forms were provided at the ends and a formed construction joint (see Fig. 3) was used at the sides of the invert pours. The form for the construction joint served as a track for a screed. Invert pours for the regulating tunnels were vacuum treated to remove excess water and then finished by light troweling to remove fins left by the vacuum process. Prior to positioning the collapsible steel arch forms, any fallen Fort Union caught by the protective wire mesh in the crown was removed together with any loose blocking. Lagging also was removed, where necessary, to allow free movement of concrete. Two overhead concrete lines, or slick pipes, were used for the arch and side pours. Men working inside the forms distributed and vibrated the concrete except in the extreme top of the arch where the slick pipes were buried in the concrete and withdrawn slowly as placing progressed. As an aid to placing the 6-inch slump concrete, air was injected at intervals into the discharge line. This air slugging was held to a minimum in order not to distort the steel forms or blow out the wooden bulkheads at the end of each pour. In the regulating tunnels absorptive form lining of 10 ounce duck was used below the springline while the concrete above the springline was placed directly against the forms. Forms generally were stripped 24 hours after placing was completed and the concrete was cured for 14 days by water sprays.

The upper 120 degree portion of the crown was grouted with mortar grout through grout pipes placed during construction (see Fig. 3) to fit the overbreak pattern. In general, only a small quantity of grout was required indicating that the overbreak voids in the uppermost part of the crown had been adequately filled with concrete. It was originally planned to grout in two stages with the second stage being a neat cement grout; however, when early results of the second stage grouting proved the grout take to be minor, the second stage grouting was discontinued except for about 100 feet at the main grout curtain. A decrease in grout take when the interval between the first and second stage grouting was increased indicated that swelling of the Fort Union may have filled voids left after the first stage grouting.

Construction Order and Progress

Tunnel 7 was mined first, primarily because its portal was completed first. Tunnel 5 was mined next in order to measure the effect of mining an adjacent tunnel past the test section portion of tunnel 4. Tunnels, 2, 8, 6, 4, 1, and 3 were mined in that order with mining generally being underway in two tunnels at the same time. As a result of the order of mining, widely different loading conditions occurred on the concrete linings. Tunnels 2, 5, and 7 were subjected to mining of an adjacent tunnel on both sides after being lined. This was termed the multiple tunnel case. Except for the test section portion, tunnel 4 was mined alongside the already lined tunnel 5 and after being concreted was subjected to the adjacent mining of tunnel 3. Tunnels 3 and 6 were mined between two previously concreted tunnels and the two end tunnels, 1 and 8, were mined adjacent to a previously concreted tunnel.

Construction of the eight tunnels, together with the downstream portals and connecting retaining walls, was performed by S. A. Healy Company and Material Service Corporation under a joint venture contract amounting to $14,275,216. Notice to proceed was issued on March 29, 1949 and the entire

work was completed on June 1, 1951. During the summer of 1949, the contractor assembled his plant; constructed his shops, yard, and aggregate storage facilities; and started actual construction on August 15 with the initial excavation for portal 7. Mining was started in tunnel 7 on December 10 and the tunnel was holed through on February 29, 1950. Concurrently with portal construction and tunnel lining, mining continued on two or more tunnels concurrently and all mining was completed in January 1951. After the preliminary operations of opening the portal and erecting the jumbo were completed, mining progress averaged 20 feet per day for three 8-hour shifts for the five 35 foot bore tunnels, 22 feet per day for the 31.5 foot bore of tunnel 6, and 26 feet per day for the 27 foot bore of tunnels 7 and 8. Figure 6 shows the downstream portals in July 1950 at about the midpoint of construction.

Instrumentation

The main objective of the tunnel test section program was to provide design data for the main tunnels. It was planned as a method of approaching such problems as:

(1) Magnitude and distribution of external pressures,
(2) Rate of pressure development,
(3) Possibility of pressure relief by allowing yield during construction, and
(4) Load transfer due to mining adjacent tunnels.

To develop these design data, the test section and main tunnel observation programs were planned to include the following types of measurements.

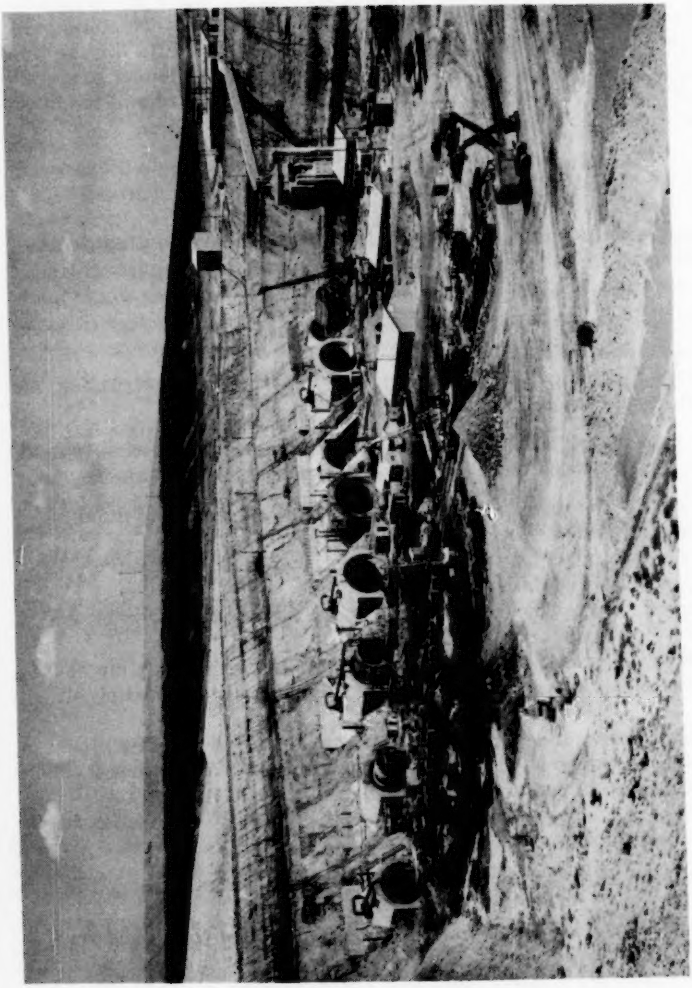
(1) Determination of stress in the steel tunnel ribs by means of strain measurements with mechanical strain gages.
(2) Measurements of changes in shape of the tunnel ribs and concrete lining by means of a precise tape extensometer.
(3) Measurements of earth movements and pore water pressures around the bore of the test section.
(4) Measurement of earth pressures on the slotted concrete section of the test section and strains in the concrete reinforcing steel, and steel ribs.
(5) Miscellaneous measurements such as compression of crusher blocks at rib joints, movement of moisture around the bore of the tunnel, and movements of the tunnel and surrounding soil.

Arrangement of Test Section

As shown in Fig. 7, the test section portion of tunnel 4 was arranged in four main sections.

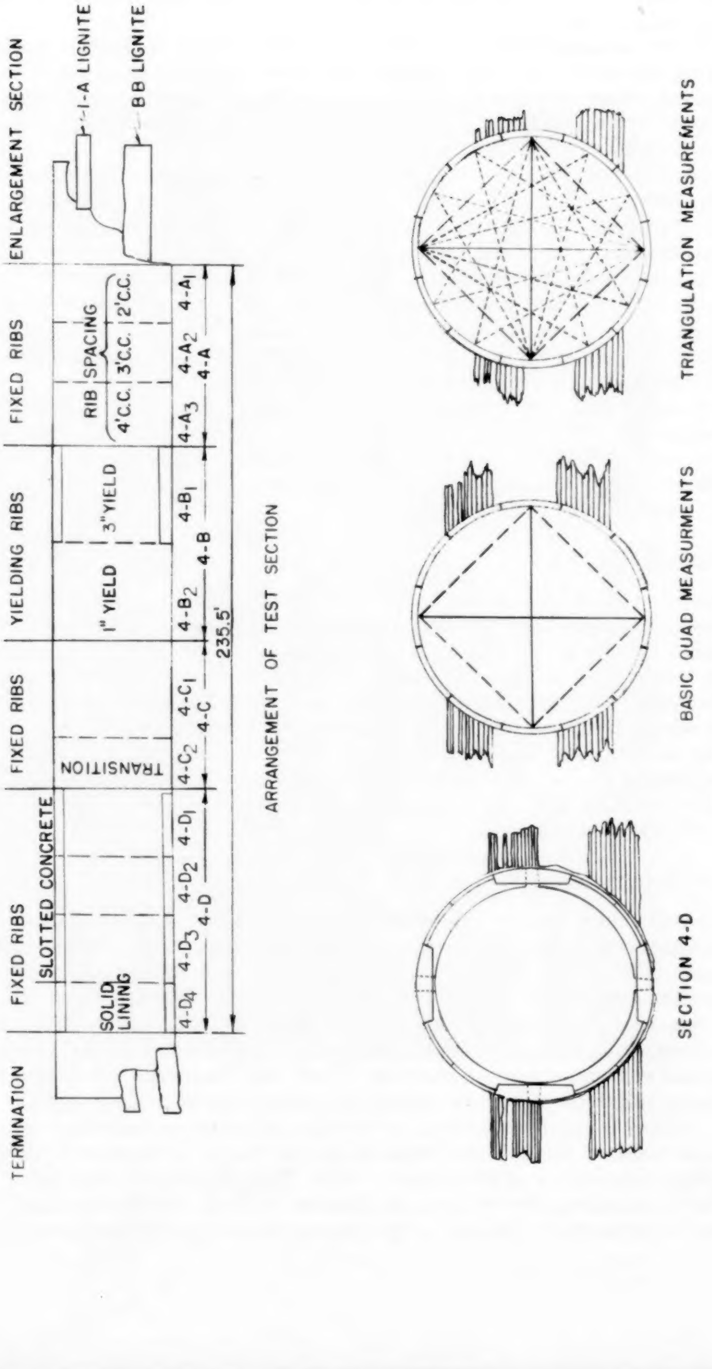
Section 4-A was supported by fixed steel ribs with bolted joints and was designed to allow yield of the Fort Union through square open spaces between lags arranged so the space between lags was approximately equal to the space between ribs. This section was divided into three subsections with the ribs spaced 2, 3 and 4 feet on centers.

Section 4-B was a yielding rib section with redwood blocks placed between the rib segments at each joint so that crushing of the blocks would allow the rib to yield. The 4-B section was divided into two subsections arranged to allow different amounts of yield. Section 4-B_1 had thick crusher blocks



DOWNSTREAM PORTALS JULY 1950 MINING TUNNELS 6 AND 8

FIGURE 6



LAYOUT OF TEST SECTION

FIG. 7

between rib segments which would allow about 3 inches of radial yield and section 4-B_2 had thinner crusher blocks to provide radial yield of about 1 inch. It was anticipated that the blocks would crush about one-half of their thickness under the design loads.

Section 4-C was designed as a fixed rib section similar to section 4-A but with lagging spaced so openings between lags were approximately equal to the width of a lag. Section 4-C had two subsections, one planned to be the major measuring station most comparable to the main tunnel construction and a transition section into the concrete lined section.

The concrete lined section, 4-D, was constructed with slots in the lining at the crown, invert, and springlines. These slots were spanned with heavy, 30-inch, wide flange, 210 pound, steel beams which approximated the thrust resistance of the concrete lining. Measurement of stress in the steel spanning the slots allowed the vertical and horizontal reactions on the tunnel to be weighed. Section 4-D was divided into four subsections: a transition from the steel rib section, the main measuring section, a transition section at the end of the main measuring section, and a non-slotted terminal section provided to eliminate end effects in the main measuring section.

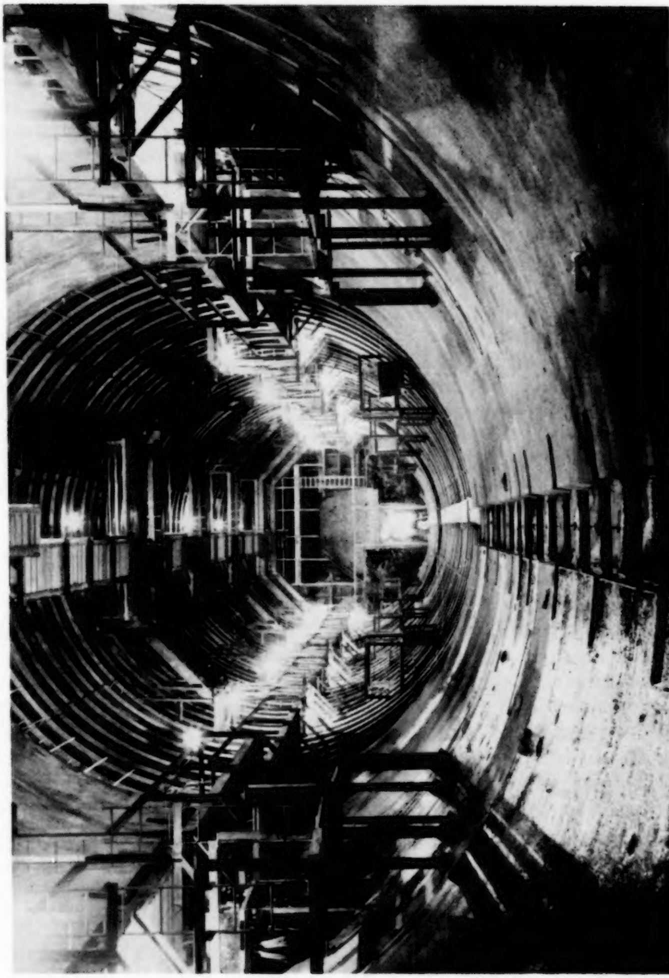
Prior to mining the test section, a shaft was sunk from the ground surface above section 4-C_2. This shaft was used later to measure ground movements and served for ventilation of the test tunnel. A system of catwalks and stairways at selected ribs in each section provided access to observation stations throughout the test tunnel as shown in Fig. 8.

Types of Measurements

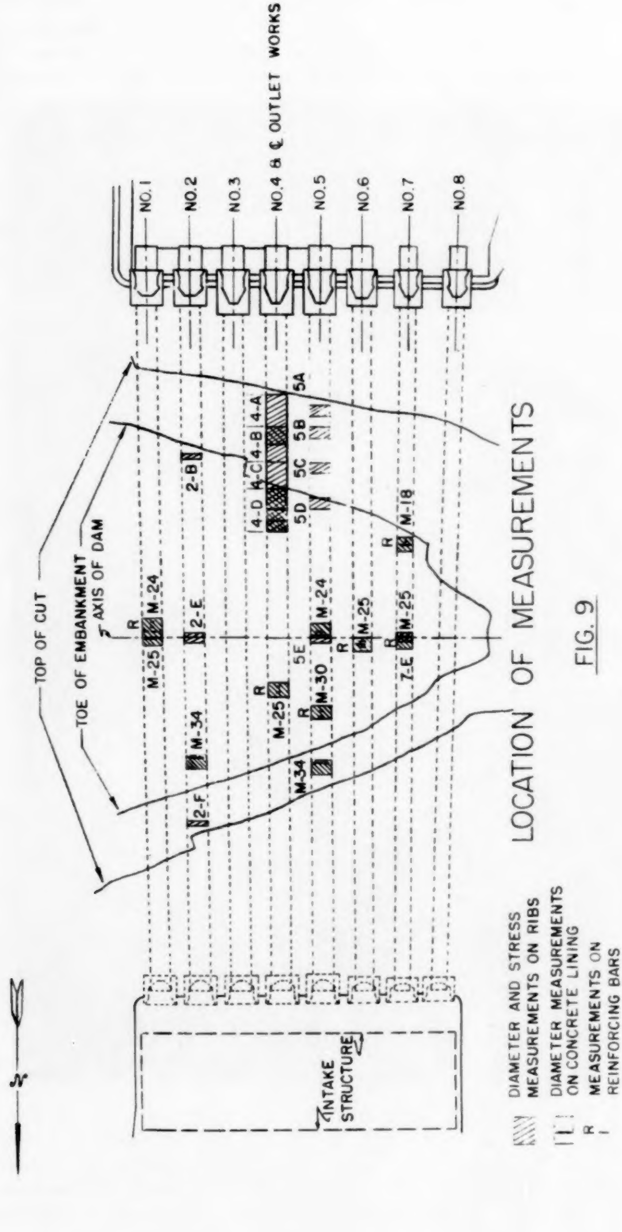
As shown in Fig. 9 mechanical measurements were made at selected locations in six of the tunnels including extensive measurements in the test section of tunnel 4. In addition electrical measurements were made in section 4-D of the test tunnel. In general, the mechanical measurements were strain measurements used to determine both strain and stress in a variety of locations. These ranged from measurement of the stretch of the main tunnels to stress determinations in the steel ribs and reinforcing steel by means of mechanical strain gages. Electrical measurements were made to determine strain in concrete and steel members and to determine pressure for the Fort Union against the tunnel bore.

Mechanical Measurements

Throughout the test section investigation, the most useful mechanical measurements were strain measurements with 2 inch and 10 inch Whittemore strain gages. From these strain measurements stresses in the steel ribs were determined throughout the test tunnel as well as at selected locations in tunnels 2, 5 and 7. In addition, stresses were determined in the reinforcing steel at selected locations in the concrete linings of several of the tunnels. Horizontal and vertical diameter changes of both the temporary rib supports and the concrete linings were determined throughout the test tunnel and at selected locations in the main tunnels with a special extensometer tape especially manufactured for this investigation by the Corps of Engineers, Ohio River Division Laboratory at Mariemont, Ohio. This instrument was patterned after a measuring device used in 1934 and 1935 by the Metropolitan Water District of Southern California for precise chord measurements on the



CATWALKS FOR ACCESS TO MEASUREMENT POINTS
FIGURE 8



LOCATION OF MEASUREMENTS

FIG. 9

Fan Hill conduit of the Colorado River Aqueduct.[8] The extensometer tape was essentially a Lo-Var nickel-steel tape to which was attached a dial gage reading to 0.001 inch with an extension range of two inches. The tape was punched with holes at intervals of 2.000 inches. This allowed the tape to be secured to a frame or head containing the dial gage with a pin and clamp arrangement in precise increments of two inches. Hooks were provided on both ends of the instrument and corresponding hooks were installed at ends of diameters or chords to be measured. Constant tension was applied to the tape by means of a calibrated adjustable spring device contained in the measuring head. Hooks on the ribs were precisely formed to the same radius as the radius of the hooks on the ends of the extensometer and were chrome plated to minimize error due to corrosion and wear.

At one or two ribs in the center of each major section within the test tunnel intensive stress and shape measurements were made. Other mechanical measurements included measurements of the amount of crushing of the wood blocks between rib segments in the yielding section of the test tunnel, strain gage measurement of the slippage at the rib segment joints, measurement of the ground surface settlement over the test tunnel, heave of the test tunnel due to release of vertical load on the invert, twist of the test tunnel as adjacent tunnels were mined past, movement of the ventilation shaft at the test tunnel was mined by, measurement of the movement of the ground toward the tunnel bore by means of reference points called spearheads driven into the soil around the bore and in the headings, measurement of the pore pressure in the clay, and elongation of the tunnels due to yield toward the excavations.

Electrical Measurements

Two types of electrical instruments were included in the test tunnel instrumentation, pressure cells for direct measurement of stress against the concrete lining and strain meters for measuring strain in the concrete lining and in the reinforcing steel and steel ribs. Two types of pressure cells were used, the Waterways Experiment Station type[9] which utilizes SR-4 strain gages to measure the deflection of an internal diaphragm under pressure of a liquid held within a contained chamber, and the Carlson stress meter, which utilizes the Carlson elastic wire strain meter[10] to measure deflection of a similar diaphragm. Both types of cells utilize a Wheatstone bridge for making observations and require load calibration prior to installation. The earth pressure cells were mounted on the west side of test section 4-D and fairly well distributed between the crown and the invert in an effort to measure directly the stress distribution and the effect of mining adjacent tunnel 5. Carlson strain meters were used to measure strains and temperatures in the concrete lining and were placed in the concrete at various depths in the lining on both sides of the tunnel. SR-4 strain gages were mounted on the reinforcing

8. An Invar-Tape Extensometer by C. H. Heilbron, Jr. and W. H. Saylor, Civil Engineering, Feb. 1936.
9. Soil Pressure Cell Investigation, Interim Report, Technical Memorandum No. 210-1, U.S. Waterways Experiment Station, Vicksburg, Miss., 15 July 1944.
10. "Five Years Improvement of the Elastic Wire Strain Meters," by R. W. Carlson, Engineering News Record, Vol. 114, No. 20, 16 May 1935, pp. 696-697.

steel and steel ribs in positions adjacent to the Carlson strain meters.

Measurements in the Test Section

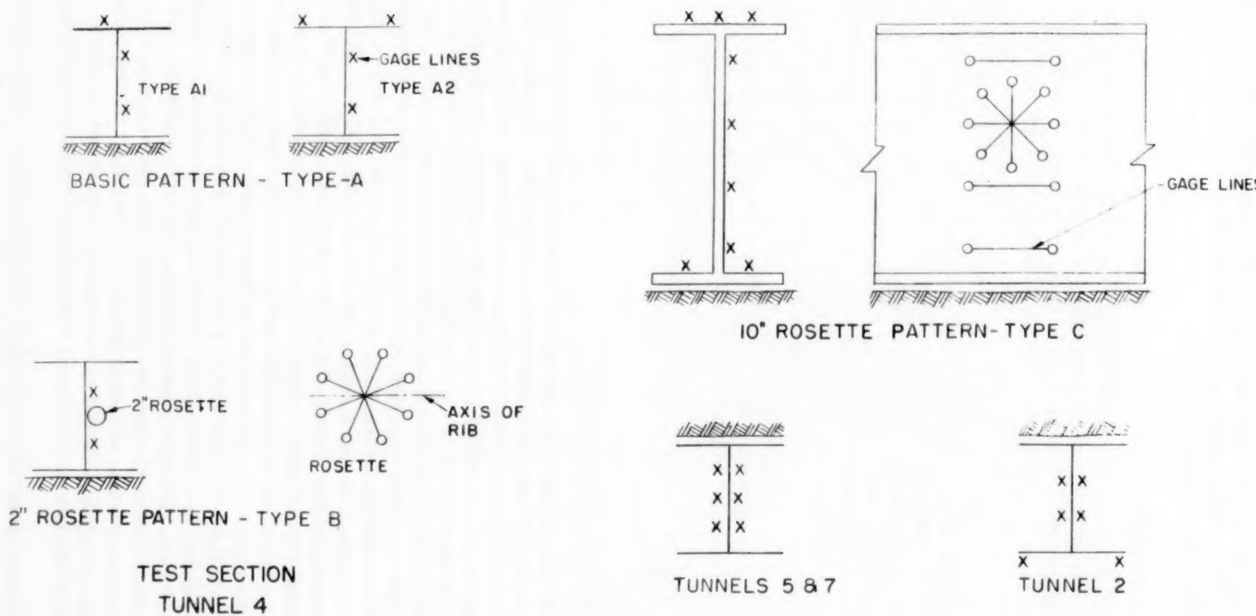
In the test section of tunnel 4 major dependence was placed on the mechanical types of measuring instruments, primarily the extensometer and Whittemore strain gage measurements. Measurements were referenced on each rib to positions corresponding to a clock face when looking into the tunnel. Thus, 12 o'clock denoted the crown location, 3 o'clock the right springline, 6 o'clock the invert position and 9 o'clock the left springline. Throughout the test section horizontal diameter measurements were made on each of the 80 ribs together with a basic pattern of 10 inch gage line measurements at the 3 and 9 o'clock positions. On every other rib through the steel rib sections additional vertical diameter and diagonal chord measurements and basic pattern strain gage measurements were made at the crown and invert. At selected ribs, additional strain gage measurements were made around the entire circumference of the rib and additional chord measurements were made in a 12 point triangulation pattern. Access to the observation points was provided by stairways and catwalks shown in Fig. 8.

Strain Gage Measurements

Stress measurements in the steel were made by Whittemore strain gages based on zero stress condition determined by initial observations on the rib segments prior to erection. The Whittemore strain gage measurements were made in three general patterns as shown in Fig. 10. The basic pattern, type A group, consisted of two 10-inch gage lines on the web and one or two 10-inch gage lines on the inside flange. The gage lines were positioned at the midpoint of the rib segment at the 3 and 9 o'clock position of all ribs and at the 6 and 12 o'clock position of all even numbered ribs throughout the length of the steel rib section. At the 3, 9, and 12 o'clock positions the flange gage line (type A 1 pattern) was offset from the axis of the rib in order to allow the Whittemore strain gage to clear the extensometer hooks. At the 6 o'clock position, two flange lines were provided, one on either side of the extensometer hook (type A 2 pattern). The type B pattern consisted of three 10-inch gage lines arranged in the same pattern as the type A group with four 2-inch gage lines forming a strain rosette on the web of the segment. These were located two to a segment on each rib segment of the six 12 point triangulation ribs in the steel rib sections. The type C pattern consisted of nine 10-inch gage lines parallel with the axis of the rib and three additional 10-inch gage lines arranged to form a rosette on the web. These groups were located on each of the heavy ribs spanning the four slots in the concrete lined section.

Early in the measurement program, it became apparent that bending of the steel ribs probably was not in the plane of the rib section. To investigate this possibility, special groups of closely spaced 10-inch gage lines were installed on five ribs. Spacing and number of gage lines of these special measurement stations varied but gage lines were generally about one inch apart in a parallel pattern around the rib. From these measurements, it was clearly established that bending occurred at an angle to the rib which varied from rib to rib.

In order to establish the probable observation error eight strain gage lines, one on each of the triangulation ribs, were measured daily for about 5 months, in exactly the same manner as routine observations. Based on these



PATTERNS OF STRAIN GAGE LINES

FIG. 10

measurements the probable order of accuracy of strain gage observations, including errors due to different instruments and different observers but excluding very infrequent erratic observations, was found to be in the order of $\pm$ 600 p.s.i. unit stress in the steel.

Extensometer Measurements

Horizontal diameters were measured on each rib to determine the trend of the rings to deform under load. Basic quad measurements were made on every other rib throughout the test section. As shown in Fig. 7, the basic quad consisted of horizontal and vertical diameters together with the four chords connecting the ends of the diameters. After the measurement program was under way, it became apparent that the high order of accuracy of the extensometer tape permitted the use of the horizontal and vertical diameters without resorting to the diagonal chord measurements so the diagonal chord measurements were discontinued. At the center of each of the main observation sections, observations were made on 22 chords arranged in the 12 point triangulation measurement pattern shown in Fig. 7 to determine deformation of the complete ring relative to the invert. Twist or rotation of the tunnel was determined by direct measurement between a plumb line dropped from the 12 o'clock position and the 6 o'clock position. From a series of daily observations at six of the horizontal diameters over a 5 months check period it was determined that individual measurements could be checked by different observers with different instruments within an accuracy of about 0.002 inch; however, the total probable error from all causes over the entire period was in the magnitude of $\pm$0.010 inch.

Miscellaneous Mechanical Measurements

While the bulk of the mechanical measurements in the test program consisted of extensometer tape and Whittemore strain gage observations, many other types of mechanical measurements were made to measure as fully as practicable the movements and deflections of the tunnel and the surrounding ground. Based on the experience in the Chicago subway tunnel tests,[11] it was considered desirable to include measurements of soil movements around the test section in the observation program. The original program of measurements included settlement observations on the ground surface above the tunnel and observations to determine movement of soil toward the heading as well as radially toward the open bore. Construction of the ventilation shaft provided an additional opportunity to measure ground movements and pore pressure changes as the tunnel approached and passed the shaft location. Twenty-two settlement points were installed in three lines on the ground surface above the test section. Standard survey equipment was used for level observations to 0.001 foot and levels were referenced to a permanent bench mark about one mile away. Ground surface settlements ranging from 0.02 to 0.08 foot were observed above the test section before the reference points were obliterated by construction of the embankment. The steel casing of the ventilation shaft was stepped down in 2 inch diameter increments from 48 inches at the top to 40 inches at the crown of the tunnel. At each slip joint

11. "Earth Pressure and Shearing Resistance of Plastic Clay," A Symposium by Karl Terzaghi, Ralph B, Peck, and W. S. Housel, ASCE Transactions Vol. 108, 1943, pages 964 to 1109.

between casing of different diameter the vertical movement of the casing sections was measured by a steel tape from the settlement point on top of the casing. Movement in the shaft continued throughout the entire observation period showing a slow, steady, downward movement of the ground ranging from 0.14 foot near the surface to 0.28 foot near the crown of the tunnel when measurements were stopped in July 1950. The deformation took the form of an elongation, or stretch, of the ground practically all of which occurred as the tunnel was mined. After mining, each subsequent loading event caused additional settlement but only a very small additional elongation in the ground. Measurements were made throughout the course of the investigation to detect heave, elongation, and twist or rotation of the tunnel. Heave was measured by survey of the tunnel invert referenced to bench marks set in the powerhouse excavation area and in the access tunnel and ranged from 0.5 foot at the portal to 0.15 foot at the north end of section 4-D measured from a zero established subsequent to excavation. In the center of the powerhouse excavation 1.5 feet of rebound due to excavation unloading were measured on gages established prior to the start of the excavation.[12] Elongation was measured by survey observations on horizontal control points established and measured incidentally to the tunnel construction and showed a slight stretch of the tunnel as the slope yielded toward the excavation. Twist was determined by plumb bob observations on the basic quad and 12-point triangulation measurement ribs and indicated a slight roll of the top of the test section away from the adjacent tunnels as they were mined. The tendency of the soil around the test tunnel to move into the bore was observed by measurements on spearheads or reference points driven or drilled into the soil surrounding the tunnel bore. Originally, it was planned to install both temporary and permanent spearheads with the temporary spearheads being those which could be driven into the soil ahead of the heading and observed for a short period of time until they were removed by excavation as the tunnel advanced. When relatively small movements were observed on a few temporary spearheads installed in the access tunnel and in the early portion of the test section, installation of temporary spearheads was discontinued. Permanent spearheads for observations over long periods of time, were installed in the ventilation shaft and in a radial pattern at two ribs in section 4-A. The initial installation was supplemented by permanent type spearheads installed at the end sections and later by additional radial spearheads installed in sections 4-B and 4-C just prior to mining tunnel 5. Spearheads indicated a continued slow plastic movement of the Fort Union toward the tunnel throughout the single tunnel case. The rate of movement was very small but during a one year period sizeable total movements were observed. As much as 0.2 foot of movement of the soil relative to the ribs was observed in section 4-A although measurements in section 4-B and 4-C indicated smaller movements. End spearheads moved in 0.04 foot at the south end and from 0.07 to 0.18 foot at the north end. As an aid in interpreting the load distribution on the ribs, the detailed pattern of blocking and overbreak was obtained by photographing the excavated surface of the entire test section after blocking. Overbreak was measured in reference to the rib generally at the rib segment joints and at segment midpoints with additional measurements at quarter points where

12. "Rebound Gages Check Movement Analysis at Garrison Dam," by K. S. Lane and S. J. Occhipinti, Proceedings, Third International Conference on Soil Mechanics and Foundation Engineering, Switzerland, August 1953.

necessary. Lignite beds were referenced to each rib. Measurement of crusher block compression between the rib segments in the yielding rib section 4-B were made with a vernier caliper or a micrometer both measuring to 0.001 inch. Lateral movement, or slippage, of the rib segment joints plates was measured on a 10-inch strain gage line installed with one gage hole on each joint plate at each joint on each 12 point triangulation rib in sections 4-A, 4-B, and 4-C. Slippage of joints was negligible. Nine double tube closed system U.S. Bureau of Reclamation type piezometers,[13] modified by the addition of a brass driving point, were installed in the Fort Union surrounding the test tunnel and the vent shaft. Six were installed in the soil between tunnels 4 and 5 and three were installed above the test tunnel from the vent shaft. Prior to mining tunnel 5 alongside the test section, six additional piezometers were installed in the soil pillar between the two tunnels and eight radial piezometers were installed with the tips about 5 feet from the bore. Piezometers installed from the vent shaft showed some pressure increase as the heading approached and passed the shaft. Two of the three piezometers in the shaft appeared to be located near drainage layers which affected the observations; however, one piezometer indicated a pressure of about 30 feet of water above the tip throughout the single tunnel case. The piezometers installed in the clay from inside the tunnel in general showed steady pressure decreases as would be expected in a consolidating material. In an effort to measure moisture movement a system of moisture sampling was set up to take periodic samples of soil at various depths from the tunnel bore at selected stations. Due to variations in soil, this program was relatively unproductive and was discontinued.

Electrical Measurements

A rather comprehensive electrical instrumentation program was accomplished in section 4-D where four types of electrical instruments were installed in the center portion of the slotted concrete section.

Earth pressure cells were installed around the west side of the tunnel from crown to invert. The first pressure cells were rigidly mounted and early observations indicated high and rapidly increasing pressures. It became apparent that these cells were acting as blocking points, attracting load which arched onto the stiff cells from adjacent unsupported zones. As pressure on the cells approached their design limit the more heavily loaded cells were relieved by releasing their supports or by adding blocking adjacent to the cells. This type of relief was temporary so subsequent cells were installed with loose bolted mounts or flexible mounts. Load increase from doming back of the heading was greatest when cells were installed close to the heading and less when cells were installed further back and indicated doming occurred within about 60 feet back of the heading. Generally, in the first month after the concrete lining was placed, there was a reduction in pressure which was striking for the rigidly mounted cells and significantly less for the loosely mounted cells. This indicated that the high stress concentrations at the blocking points tended to dissipate when full support was provided by the concrete lining. The cells quickly indicated changes in pressure due to changes

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13. "Ten Years of Pore Pressure Measurements," by F. C. Walker and W. W. Daehn, Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, June 1948.

in temperature and loading such as the addition of the embankment and the load increase as tunnel 5 was mined by. Qualitatively, the pressure cells consistently registered changes in load and often furnished the first indication of load change. From a quantitative standpoint, the results were not dependable, primarily due to eccentricities of loading for which the cells were not designed and for which they were not particularly adapted. The mortality rate of earth pressure cells was high; 2 out of 16 Carlson cells and 19 out of 22 Waterways Experiment Station cells failed although several of the failed Waterways Experiment Station type cells subsequently gave responses which seemed qualitatively reasonable. Since eccentric loading conditions are inherent in tunnels where loading is apt to be irregular and have tangential components, it was concluded that earth pressure cells used in this investigation were not suited to pressure measurements in such installations.

Duplicate installations of Carlson elastic wire strain meters were installed in groups of three at different depths through the concrete lining at locations slightly less than 45 degrees above and below the horizontal axis on both sides of section 4-D. Strains were converted to stress on the basis of modulus of elasticity determined on small cast and cored specimens of the concrete lining. Baldwin-Southwark SR-4 strain gages were installed on the reinforcing steel and steel ribs at locations near the Carlson strain meters. The Carlson strain meters operated quite satisfactorily over the entire test period; unfortunately, no "zero load" control meters were installed and the temperature corrections, as approximated from theoretical considerations, were so large that the actual stresses were masked by the corrections. Although the SR-4 gages were carefully bonded to the steel with bakelite cement and carefully water proofed with sealed metal enclosures for gages and leads, only 7 out of the 16 SR-4 gage installations remained operative after concreting was completed. None of the gages survived the entire observation period and those gages which did operate for part of the time showed extremely inconsistent results. The high mortality rate and the inconsistent results indicate that the installation technique must be further developed before SR-4 gages are satisfactory for long term field installations of this nature.

Main Tunnels

Since the extensometer tape and Whittemore strain gage observations were the most reliable and satisfactory observation methods used in the test section program, measurements in the main tunnels were confined generally to these two types although other special measurements were made as occasion required. To obtain data on the inter-effects of tunnels 4 and 5 as tunnel 5 was mined alongside the test section, measurements similar to those in the test section, but limited considerably in scope, were made in tunnel 5. As shown in Fig. 9, these measurements were made at four sections opposite corresponding sections 4-A, 4-B, 4-C, and 4-D. Additional measurements were made at the centerline of the dam in section 5-E and on ribs in tunnels 2, 5, and 7 as shown in Table 2. In tunnels 5 and 7, 10-inch Whittemore strain gage observations were made on three lines equally spaced on both sides of the web as shown on Fig. 10. In tunnel 2, where smaller rib steel was used, only two gage lines were used on the web supplemented by two additional gage lines on the inside flange. Chord and diameter measurements were made with the extensometer tape in the same manner as in the test section.

TABLE 2

Number of Ribs Observed in Each Section

Tunnel		2			5					7
Section		2-B	2-E	2-F	5-A	5-B	5-C	5-D	5-E	7-E
Tape Measurements	Horiz. Diam.	2	5	2	5	4	4	5	5	5
	Vert. Diam.	2	5	2	5	-	-	5	5	5
	8 pt. Chords	-	-	-	1	-	-	1	-	-
Strain Gage Measurements	Horiz. 3 9	2	5	2	5	4	4	5	5	5
	Vert. 6 12	2	5	2	5	-	-	5	5	5
	12 pts.	-	-	-	1	-	-	1	-	-
Spear Heads		-	-	-	Radial	-	Radial	Radial	-	-
Rib Size		8WF48	10WF72	8WF48	10WF72	10WF72	10WF72	10WF72	10WF72	10WF49
Rib Spacing ft. on centers		4	3.5	4	3	3		3	3	4

Measurements were made on the concrete linings of all the finished tunnels except 3 and 8. These measurements consisted of horizontal and vertical diameters at the locations shown in Fig. 9. As the diameter measuring stations were established during construction, the vertical diameter was rotated slightly to avoid interference with construction operations at the invert. Extensometer hooks were installed from a light jumbo; most of the horizontal diameter stations were installed immediately after construction but some of the vertical diameters were installed later when the crown was more readily accessible. Horizontal diameter measurements were made from a light magnesium ladder. The vertical diameter was measured by inserting the end of the extensometer tape into the crown hook by use of a bamboo pole with the extensometer head being read at the invert.

Measurements of stress were made on several of the transverse bars in the inner ring of reinforcing at the invert in five of the tunnels as shown in Fig. 9. For these measurements, the concrete was chipped from around the bars and two 10-inch Whittemore strain gage lines were installed on each bar. Generally, two bars 10 feet apart were observed at each measurement location. The gage lines were established at various times after concreting; some were installed after only the invert had been placed when the bars were considered to be under zero stress while others were established after the full lining had been placed and the bars already were under stress. Where gage lines were established on stressed bars the true zero stress condition was determined by cutting the bars upon completion of the observations.

The elongation of tunnel 5 due to yield toward the portals was observed by measurements between a series of cadmium plated expansion bolts originally set precisely 100 feet apart near the invert construction joint. Observations were made with a Lo-Var tape under a constant pull of 25 pounds and fully supported throughout its length by the concrete lining.

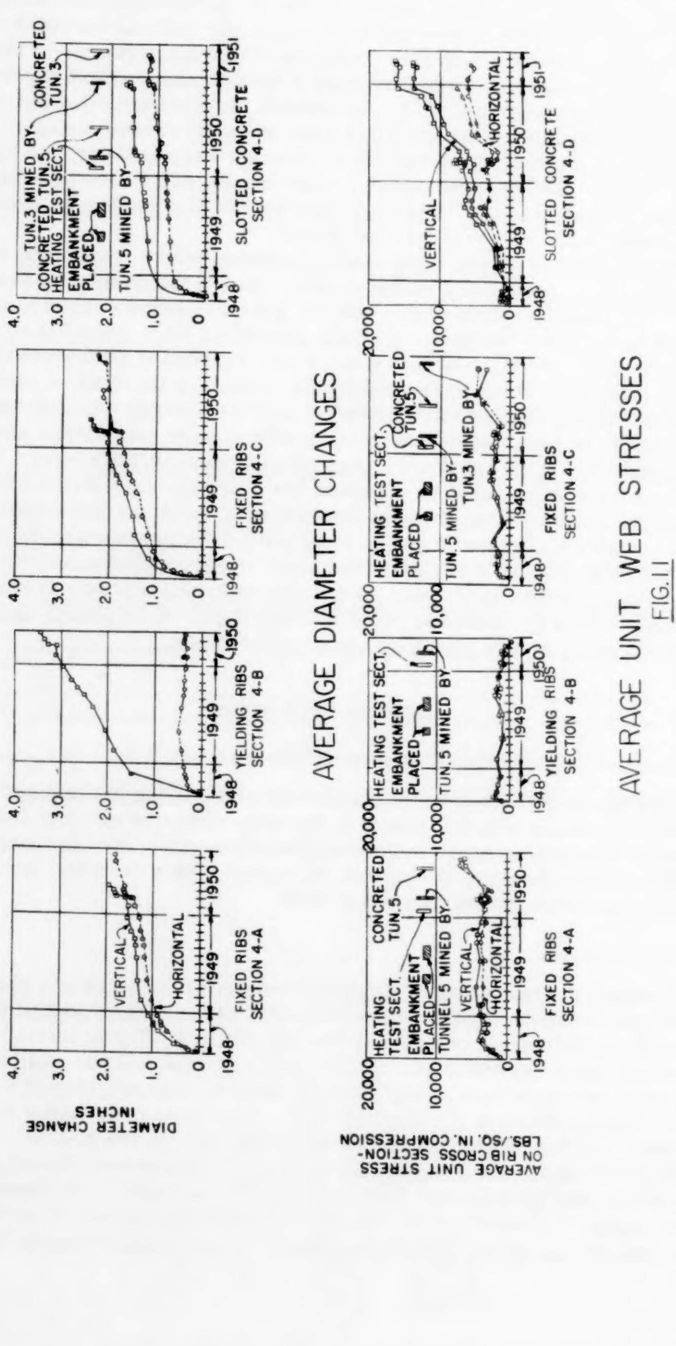
Observation Results

Test Section- Single Tunnel Case- Steel Rib Sections

During the single tunnel case, the period from mining the test section until just prior to mining adjacent tunnel 5, the only change in external load was addition of the embankment. In general, slow increases of stresses and deformations were observed throughout the period with a definite increase in rate while the embankment was being built.

Diameter Changes

Throughout the test section, vertical diameters decreases and horizontal diameters increases continued over the entire measurement period developing rapidly in each steel rib after it was set in place. The greatest deformation in each rib occurred as the heading was advanced and the soil load arched onto the rib slowing down after the heading had progressed sufficiently to allow the full effect of arching to be felt. The average diameter changes in each section of the test tunnel are shown in Fig. 11. In the yielding rib section, 4-B, the rate of crushing of the redwood blocks between rib segments exceeded the rate of diameter change from external load. The circumferential shortening due to crushing of the blocks increased the amount of vertical diameter shortening and decreased the amount of horizontal diameter lengthening.



AVERAGE DIAMETER CHANGES

AVERAGE UNIT WEB STRESSES

FIG.11

Thus, large vertical diameter decreases and relatively small horizontal diameter decreases were observed. In all other sections the vertical diameter shortening was only slightly greater in magnitude than the horizontal diameter lengthening.

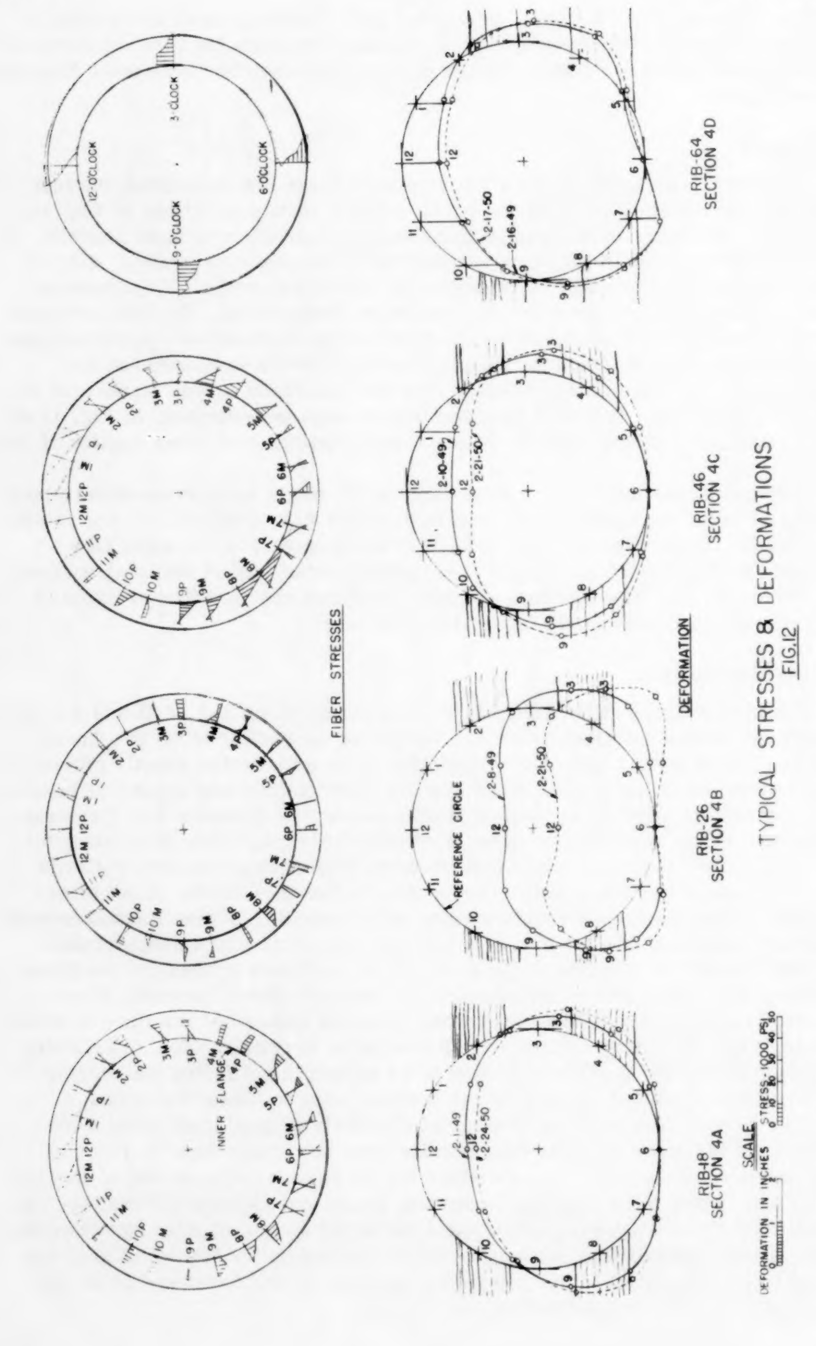
Stresses

In general, stresses in the steel ribs were quite low throughout the test section and especially low in the crusher block section as shown in Fig. 11. Stresses increased most rapidly immediately after the ribs were erected. A noticeable increase in stress was observed in the concrete section, 4-D, in the summer of 1949 when the embankment was constructed. This increase was not noticeable in the more flexible steel rib sections. The web stresses in the steel rib sections were nearly equal on the vertical and horizontal axes while in the concrete section, 4-D, the vertical stress (measured at the springline) was substantially higher than the horizontal stress (measured at crown and invert). It should be noted that the unit web stresses in Fig. 11 do not reflect load directly due to differences in spacing and cross section of the steel ribs.

Typical stress distribution at the end of the single tunnel case determined at the 12 point triangulation stations in sections 4-A, 4-B and 4-C are shown in Fig. 12. In sections 4-A and 4-C, average moments in the steel ribs roughly paralleled the development of thrusts; about half of the total moment of 200 to 250 inch kips per foot of tunnel occurred shortly after erection of the ribs with subsequent increase at a slow rate.

Stress Distribution

The stress distribution shown in Fig. 12 for sections 4-A, 4-B and 4-C indicate the stress distribution across the rib cross section to be non linear and non symmetrical. Since the steel ribs were not erected exactly plumb, erection stresses were introduced into the observations and tipping or bending in the direction parallel to the tunnel axis introduced stresses into the measurements which were thought to be of considerable magnitude in relation to the total rib stress. The gage lines on most ribs were grouped in patterns covering only a portion of the cross section of the steel so the effect of eccentric loading on the observations was indeterminate. Stress measurements made at close intervals on five selected ribs indicated that a considerable variation could be expected in the distribution of stress across the rib cross section. Eccentric stress distribution has been attributed partially to eccentric loading and partially to blocking stresses induced at the time of erection together with other irregularities inevitable in construction. Variation in properties of the steel was thought to be an additional factor influencing the variation in stress across the rib section. Due to these variations, average results from a large number of similarly located gage lines were considered far more reliable than results from any single line or group of gage lines. Average gage line stresses for all gage lines in an entire section of the test tunnel were used for computing thrust and moment for that section. In the steel rib sections the thrusts and moments were computed by assuming linear stress distribution across the rib as defined by the two web lines; the flange line stresses were disregarded since they were considered to be the most influenced by cross bending.



TYPICAL STRESSES & DEFORMATIONS
FIG.12

Shape Changes

Shape changes for typical twelve point triangulation ribs within the steel rib sections are shown in Fig. 12 for two dates during the single tunnel case. Deformation has been plotted to an exaggerated scale based on the assumption that the 6 o'clock position remained in place and that the 12 o'clock position moved on the diameter between points 12 and 6. The minor slippage at the rib segment joints was ignored in computing the shape changes.

Test Section—Single Tunnel Case—Concrete Section

In section 4-D, it was assumed that the low stresses, moderate moments, and relatively large diameter changes measured on the temporary steel ribs prior to concreting were not transferred to the concrete lining. Thrust, moment, and diameter change of the concrete lining increased gradually with time. Larger increases followed major load changes such as construction of the embankment and mining of the adjacent tunnels. Typical deformation and stress distribution in the steel beams spanning the slots in the concrete just prior to mining tunnel 5 are shown in Fig. 12. Average diameter changes and average web stresses are shown in Fig. 11. The deformation prior to February 1949 was the deformation of the steel ribs, thus the deformation of the concrete lined section was relatively small. On the other hand, the moment in the steel spanning the slots increased considerably after concreting, particularly during construction of the embankment, and reached a value of about 1500 inch kips per foot of tunnel in the crown and invert just prior to mining tunnel 5. In general, the moments at springline slots were about 500 kips per foot of tunnel lower than moments at crown and invert. After concreting, the horizontal diameter change approximately equalled the vertical diameter change and equalled about 1/2 inch when measurements were stopped in the spring of 1951. Throughout the investigation, the horizontal load was about half of the vertical load. When observations were discontinued, the vertical load was approximately equal to the overburden load.

During the early stages of the investigation it was believed that temperature changes in the test tunnel would be small and would have only a negligible effect on the measurements. When tunnel 5 was mined alongside it became apparent that temperature changes had a considerable effect on the concrete lined section. During the winter of 1948-1949, rib stresses were observed to decrease as the air temperature dropped. The decrease in stress due to temperature decrease was observed again in the late fall of 1949 and stresses in the ribs were observed to increase in January 1950 as the tunnel was heated preparatory to concreting section 4-B_1. The effect of temperature on the tunnel seemed to be twofold:

(1) Expansion or contraction of the concrete lining due to a rise or fall of temperature which caused a corresponding increase or decrease in the total thrust on the lining, and

(2) Bending stresses resulting from a temperature gradient through the concrete lining. (As a corollary to the temperature study, it was determined by measurements that there was no temperature differential across the steel ribs spanning the slots. Any such gradient would have complicated the measurement of stress since the stress was determined by comparing gage line lengths with the length of a standard steel bar which was generally placed nearer to the inside flange than to

the outside flange and thus reflected air temperature rather than the soil temperature at the outside of the bore.)

Over the entire investigation period, the long time trends minimized the effects of local or short term temperature variations and appeared to be entirely reliable; however, at times temperature changes accompanied load changes and confused the effect of load change. For example, as tunnel 5 was mined alongside, the temperature in the test tunnel was falling. As a result considerable less thrust was measured than would have been the case had the temperature remained constant. While the effect of temperature on the measurements seemed quite apparent, the problem of correcting the data to allow for temperature variation was not readily susceptible to exact analysis with the data available.

As the temperature rose and expanded the lining, the increasing change in horizontal diameter was accentuated and the decreasing change in vertical diameter was lessened. Conversely, for a temperature drop the horizontal diameter change was lessened and the vertical diameter change was increased. In plots of horizontal and vertical diameters, it became especially noticeable that temperature drops cause a divergence of the plots. This lead to development of the difference functions shown in Fig. 13, using correct algebraic signs so that temperature effects are indicated whenever both curves move, roughly parallel, in the same direction. The algebraic difference between the horizontal and vertical diameter change plots tend to eliminate the effect of temperature change.

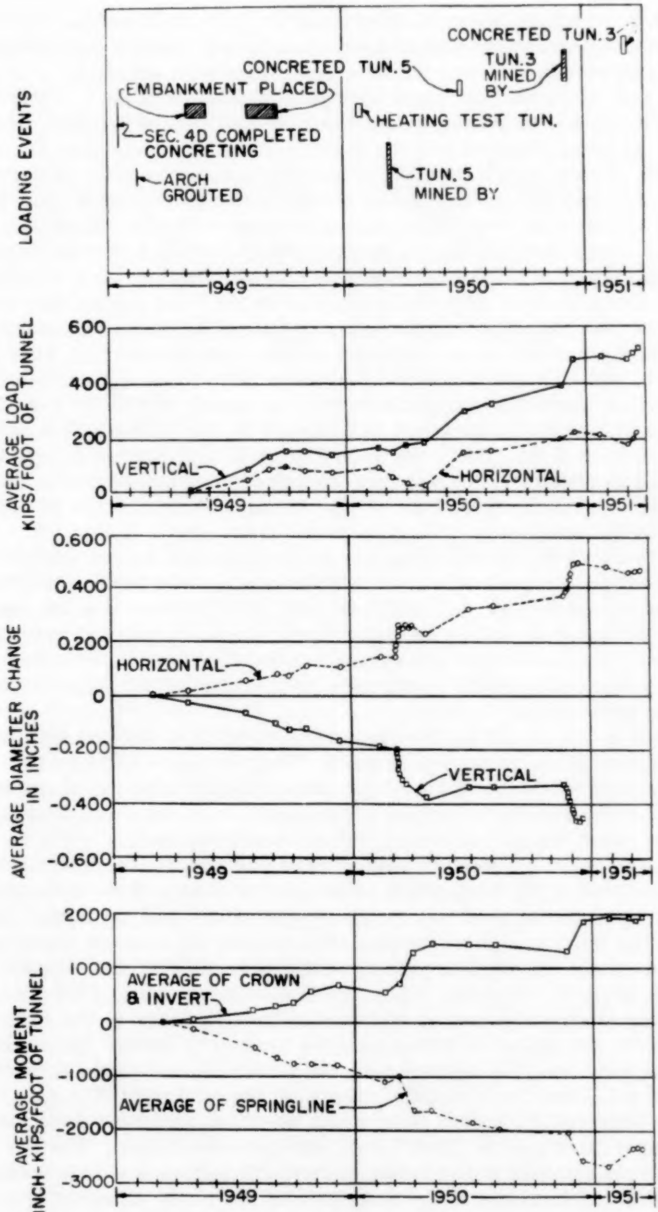
As shown in Fig. 13, both the horizontal and vertical loads tended to increase throughout the investigation period as did the difference between the two loads; however, a temperature drop would cause a contraction of the concrete lining which would decrease both the horizontal and vertical loads. Increases in vertical and horizontal loads would be observed for an increase of temperature.

The effect of temperature change on moments was similar to the effect on thrusts and diameter changes although the effect of temperature change on moment probably is complicated by the gradient through the lining when subjected to different inside and outside temperatures. In the sign convention used the crown and invert moments are positive and the springline moments are negative. Thus, the moment difference function is equal to the numerical difference between the two curves shown in Fig. 13. As with the thrust and diameter difference the moment difference function appeared to be a better indicator of the effect of load change than the moments at individual slots.

The readily apparent similarity of the differences between the two curves of each plot on Fig. 13 seemed to offer a simple direct approach to the problem of temperature effects and is covered further in the companion paper by Mr. Lane.

Test Tunnel—Multiple Tunnel Case

As tunnel 5 was mined alongside tunnel 4 in early March 1950 followed by mining tunnel 3 in late November, the multiple tunnel case was developed and extensive observations were taken in the test section in tunnel 4 as well as in the adjacent tunnels to develop as much information as possible. It was anticipated that the load increase on the tunnel 4 and on the soil pillar between tunnels due to mining the adjacent tunnel might be quite high, so very frequent measurements were made in the test section as tunnel 5 approached in

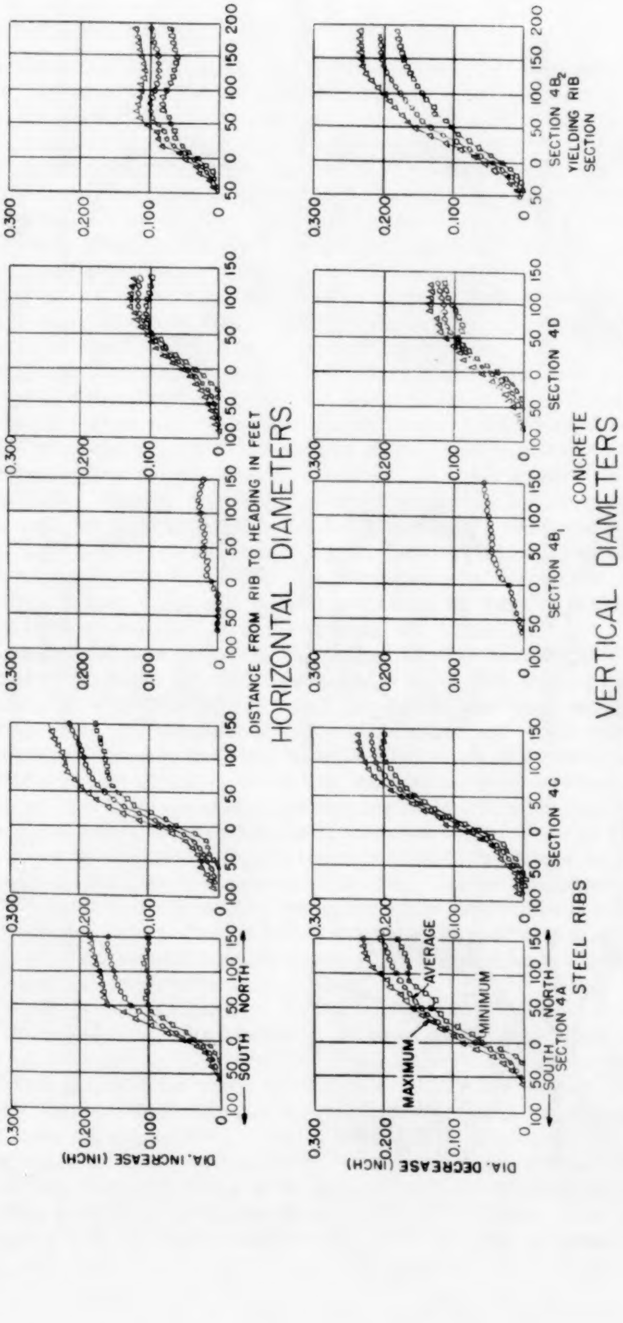


CHANGES SINCE CONCRETING SECTION 4-D
DEVELOPMENT OF DIFFERENCE FUNCTIONS
FIG.13

an effort to detect any signs of overstress as early as possible. The frequency of observations was decreased considerably when it became apparent that the increase in load was not as great as had been anticipated. Diameter changes gave the most noticeable indication of additional load. Plots of diameter change due to mining tunnel 5 versus longitudinal distance from the observation point in tunnel 4 to the heading in the adjacent tunnel 5 are shown in Fig. 14. About one month after mining the adjacent tunnels past steel rib sections 4-A and 4-C the horizontal diameter change averaged about 0.20 inch and the vertical diameter change averaged about 0.27 inch. Diameter shortening due to crushing of the blocks in the yielding section 4-B_2 resulted in a horizontal diameter change considerably smaller than the vertical diameter change. About 10 days after the heading in tunnel 5 had passed this section, the rate of crushing of the blocks began to exceed the rate of diameter change and the horizontal diameter decreased slightly. In the concrete lined sections, 4-B_1 and 4-D, the horizontal diameter change was approximately equal to the vertical diameter change; however, the change in section 4-B_1 was about half of the change observed in section 4-D, due to the stiffer concrete lining in section 4-B_1. The lining in section 4-B_1 was approximately 48 inches thick while in section 4-D it was only 36 inches thick and somewhat more flexible because of the four slots. Shape changes at each of the triangulation ribs in the test section due to mining tunnel 5 are shown in Fig. 15.

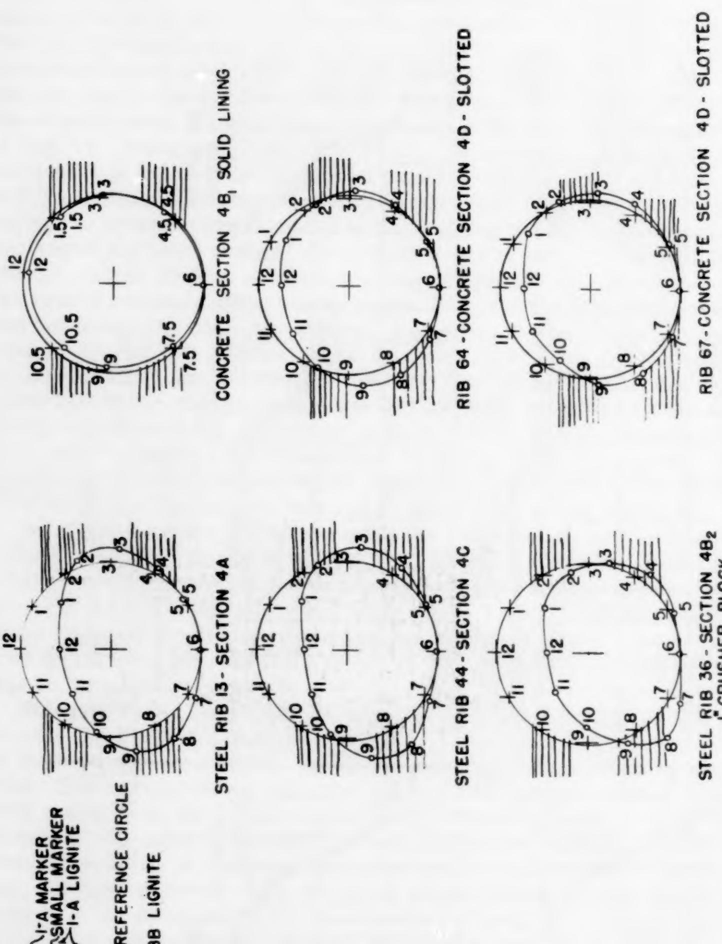
When tunnel 3 was mined alongside the test section, measurements had been discontinued except in section 4-D; however, a few horizontal diameter measurements in section 4-C and horizontal and vertical diameter measurements in section 4-B_1 before and after tunnel 3 was mined past were in the same range as the diameter changes due to mining tunnel 5 indicating the load was thrown onto the test section by mining each of the adjacent tunnels was in the same range.

Stresses in the beams across the springline slots of section 4-D increased about 4,000 p.s.i. due to mining tunnel 5. The increase was about two-thirds of the total unit stress from the single case including addition of embankment load. The corresponding increase in horizontal load was about 2700 p.s.i. In both cases, after the initial mining had increased the load, a slight decrease in stress was noted followed by a later substantial increase. The first increase occurred in the first month after start of mining in the adjacent tunnel, the drop in stress occurred in the second month, and the larger rise occurred in the third and fourth months after mining the adjacent tunnel alongside. In the steel rib sections, the increase in vertical stress was about 3000 p.s.i. while the increase in horizontal stress was about 2700 p.s.i. Here again a drop in stress occurred after the initial increase. In the yielding section, 4-B_2, the indicated stress changes were very small. As tunnel 3 was mined alongside concrete section 4-D, the vertical unit stress increased about 3200 p.s.i. and the horizontal stress increased about 900 p.s.i. These increases occurred during the first month after mining tunnel 3 concurrent with a temperature rise of about 5 degrees in the test tunnel. The few measurements obtained in the crown of steel rib section 4-C indicated a horizontal stress increase in the magnitude of 600 p.s.i. Observations showed the vertical stress increased more at the springline nearest the tunnel being mined alongside and the horizontal stress measured at the crown was larger than at the invert. The stress effect of mining each adjacent tunnel was approximately the same although the falling temperature in the test tunnel as tunnel 5 was mined delayed measurement of the full load increase



DIAMETER CHANGES IN TEST SECTION
DUE TO MINING TUNNEL 5

FIG.14



SHAPE CHANGES DUE TO MINING TUNNEL NO.5

FIG.15

until about four months after mining when the temperature in the test tunnel rose to equal the temperature prior to mining the adjacent tunnel.

Results of Observations on Ribs in Other Tunnels

Observations on the steel ribs of tunnel 5 together with similar observations in tunnels 7 and 2 confirmed the earlier measurements in the test section of tunnel 4. Relatively small average web stresses and diameter changes were measured in most cases, although some rather large moments with resulting high outer fiber stresses were found on a few ribs in each tunnel. In no case was evidence of crumpling or actual rib distress noted even though stresses in the range of the yield point were observed. As in the test section, total stresses were observed with the zero readings taken prior to rib erection. On the other hand, diameter and chord measurements did not include some of the early movements since the initial diameter measurements usually were made on the Sunday following erection. In tunnel 2, initial measurements were made as much as a month after erection.

Maximum diameter, stress, and moment changes in each tunnel compared with similar observations in the test section are given in Table 3. In the main tunnels steel ribs were used as temporary support from three to four months, averaging about 3-1/2 months so only the data for the first four months are given for sections 4-A and 4-C in order to provide a useable comparison. Tunnels 2, 4, and 7 as well as section 5-E, represent the single tunnel case while sections 5-A and 5-C represent a temporarily supported tunnel alongside the completed test tunnel which had both temporary and permanent supports at the time the measurements were made. Results of measurements in tunnels 2, 5 and 7 are shown as plots of average diameter change, web stress, and moment on Figs. 16, 17 and 18.

Diameter changes in tunnel 5 averaged about 0.5 inch which was less than the average of about 1.0 inch (see Fig. 11) in the test section at comparable periods after erection even though lighter steel was used in tunnel 5. The difference was probably due to early rapid movement which was not included in the observations in the main tunnels. The average thrusts, expressed as average unit web stresses, were low, varying between 4,000 and 8,000 p.s.i. Moments also were generally low ranging from 0 to 1000 inch kips; however, on two ribs near the axis of the dam stresses in the range of 30,000 p.s.i. were found on the inside flanges (extropolated to the flange on the assumption of linear stress distribution across the rib). Radial spearheads in sections 5-A, 5-C, and 5-D indicated slight movements toward the bore ranging generally from 0.02 to 0.04 foot with a maximum of 0.08 foot measured relative to the ribs. Movement usually occurred at a rather uniform rate within the first month after installation.

Diameter measurements in tunnel 7 indicated very little change for the light rib steel which was spaced on 4 foot centers. Due to the flexibility of the light rib support stresses in the ribs were relatively low ranging from 2,000 to a maximum average of less than 6,000 p.s.i. The horizontal and vertical thrusts were about equal with corresponding low moments. Outer fiber stresses extrapolated from the web stresses ranged from 16,000 to 29,000 p.s.i. in compression and 3,700 to 4,900 p.s.i. in tension.

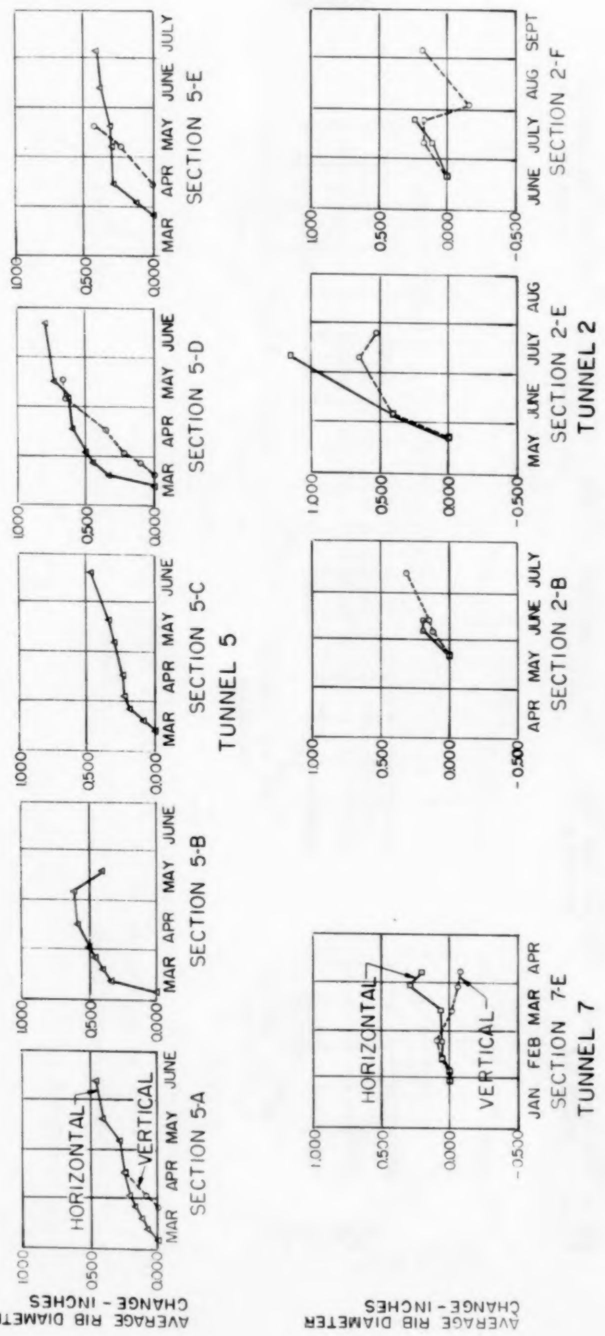
Average diameter changes in tunnel 2 were small except in section 2-E. The maximum horizontal diameter changes at individual ribs were 0.35 inch in section 2-B, 1.0 inch in section 2-E, and 0.35 inch in section 2-F. The

TABLE 3

SUMMARY OF MEASUREMENTS
STEEL RIBS DURING TEMPORARY SUPPORT
PERIOD

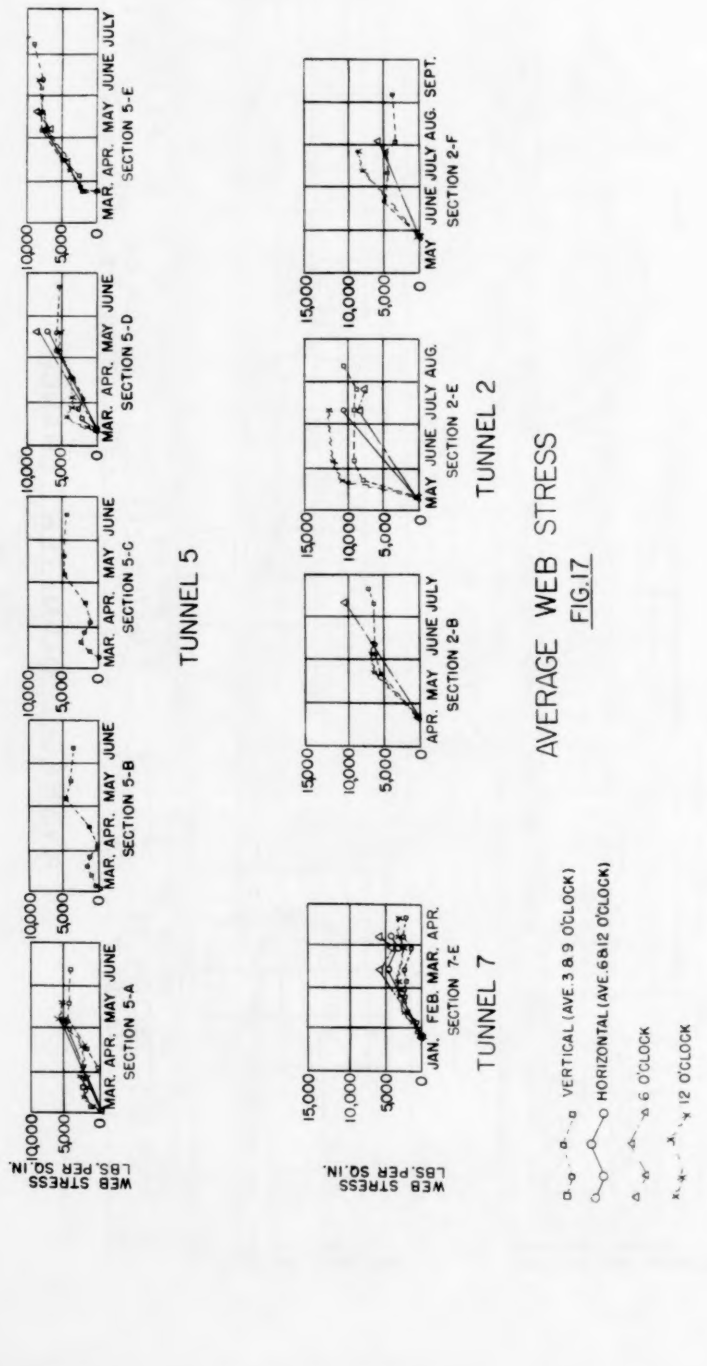
Section		2-B	2-F	2-E	7-E	5-A	5-B	5-B	5-D	5-E	4-A	4-C
Rib Spacing (ft)		4	4	3½	4	3	3	3	3	3	3	3
Bore Diam. (ft)		35	35	35	27	35	35	35	35	35	36	36
Rib Size		8WF48	8WF48	10WF72	10WF72	10WF72	10WF72	10WF72	10WF72	10WF72	12WF99	12WF99
	Clock Position											
Max. Web Stress Compression (1)	3	6,600	6,000	10,400	3,300	4,400	5,000	5,300	5,800	8,300	4,070	2,160
	6	7,200	4,600	10,400	2,200	4,400	3,100	4,600	4,800	9,700	3,820	2,250
	9	12,400	5,900	16,100	5,300	5,300	-	-	8,300	8,600	3,300	2,160
	12	6,800	9,000	12,500	3,600	5,700	-	-	5,800	8,300	3,980	2,160
Max. Thrust Kips/Lin. ft. of tunnel (1)	3	23.3	21.2	62.9	17.5	31.1	35.3	37.4	41.0	58.6	39.5	21.0
	6	25.4	16.2	62.9	11.7	31.1	21.9	32.5	33.9	68.5	37.1	21.8
	9	43.8	20.8	97.4	28.1	37.4	-	-	58.6	60.7	32.0	21.0
	12	24.0	31.8	75.6	19.1	40.2	-	-	41.0	58.6	38.6	21.0
Max. Moment Inch Kips/ Lin. Ft. of Tunnel (1)	3	-60	-110	-100	+80	+60	+80	-150	+90	-70	+191	+153
	6	-170	-50	-90	+80	+230	-160	+130	-40	+180	-175	-197
	9	-120	+280	+780	-60	+280	-	-	+360	-320	+155	+225
	12	+190	+260	+360	+200	+280	-	-	+270	+300	+292	+350
Max. Outer Fiber Stress p.s.i. (2)	Measured	40,000	+2600	+39,100	-	-	-	-	-	-	+8,700	+20,000
	Computed	28,600	+40,000	+40,000	+26,500	-30,000	-	-	+40,000	-35,000	-14,000	-14,000
	Clock Position	6	6	6	12	6	-	-	6	6	6	3
Max. Dia. Change for Section (Inch)	Hor.	+0.3	+0.2	+0.7	+0.1	+0.5	+0.6	+0.5	+0.8	+0.4	+0.9	+1.1
	Vert.	-0.2	-0.2	-1.1	-0.3	-0.2	-	-	-0.7	-0.4	-1.1	-1.4

\+ moments means couple increases radius of rib + stress means tension
\- " " " decreases " " " - stress means compression
(1) Average of all measured ribs in the section.
(2) Approximate for individual rib showing maximum in the section.



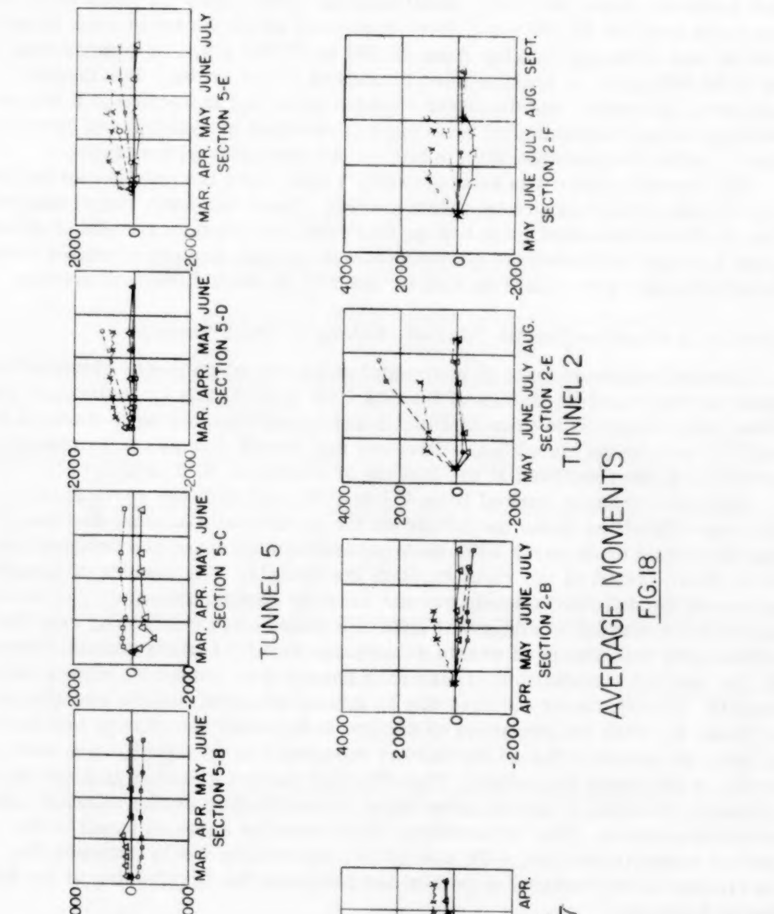
AVERAGE DIAMETER CHANGE

FIG. 16



AVERAGE WEB STRESS

FIG. 17



AVERAGE MOMENTS

FIG.18

The corresponding maximum vertical diameter changes were 0.25 inch in sections 2-B and 2-F and 1.35 inch in section 2-E. Average unit web stresses and moments were relatively small although outer fiber stresses at or over the yield point of 39,000 p.s.i. were measured in the invert of each measuring station and stresses ranging from 21,000 to 37,000 p.s.i. in compression and up to 24,000 p.s.i. in tension were measured at the crown. The largest stresses, moments, and diameter changes occurred in section 2-E where seepage caused loosening of blocking and resulted in unsupported spans of 15 feet or more between blocking points on the west side of the tunnel.

The extreme fiber stresses shown in Table 3 are the maximums for individual ribs within each observation section. Since the main tunnel observation sections contained from two to five ribs, the average results of observations are less reliable than observations in the test section of tunnel 4 where measurements were made on nine or ten ribs in each individual section.

Results of Observations on Concrete Lining of Other Tunnels

Limited measurements of horizontal and vertical diameter changes were made on the concrete linings of tunnels 1, 2, 4, 5, 6, and 7. Settlement profiles were observed in tunnels 1 and 5 and measurements were made in tunnel 5 to determine elongation. Stresses and stress changes were observed in reinforcing bars exposed at the inverts of tunnels 2, 4, 5, and 7.

Diameter changes ranged from 0.1 to 0.35 inch with the vertical diameter decrease about the same magnitude as the horizontal diameter increase during the period up to early 1953 when measurements were discontinued just prior to diversion of the river through the tunnels. The results of measurements on the different tunnels are not directly comparable since the measurements were started at somewhat different times after concreting and thus reflect only the effects of events subsequent to initial observations. Undoubtedly, the largest contribution to diameter change was caused by mining adjacent tunnels. The diameter changes due to mining adjacent tunnels are compared in Table 4. With the exception of section 4-B_1 where the change was distinctly less, presumably due to the thicker concrete lining, the changes were generally of the same magnitude. The effect of contact grouting at a maximum pressure of 50 p.s.i. was so minor that it was masked by the effect of temperature changes. The temperature effect was the same as found in the slotted concrete section, 4-D; a drop in temperature would increase the shortening of the vertical diameter and decrease the lengthening of the horizontal diameter.

A reinforcing bar in monolith 18 of tunnel 7 was found to be under a tension stress of 7,000 p.s.i. which represented the combined effect of mining both adjacent tunnels. In tunnel 5 an increase in bar stress of about 12,000 p.s.i. and in tunnel 4 an increase of about 15,000 p.s.i. were noted upon mining one adjacent tunnel. Total unit stress in the bars ranged up to 36,000 p.s.i. and was significantly affected by temperature changes. A portion of the observed stress due to temperature change was caused by the method of observation where the exposed section of the bar acted partly as if held by fixed supports at each end. The effect of temperature change was probably much greater in the exposed test bars than in normal embedded bars. The technique of measuring stresses on exposed bars is considered applicable only where temperatures can be maintained constant.

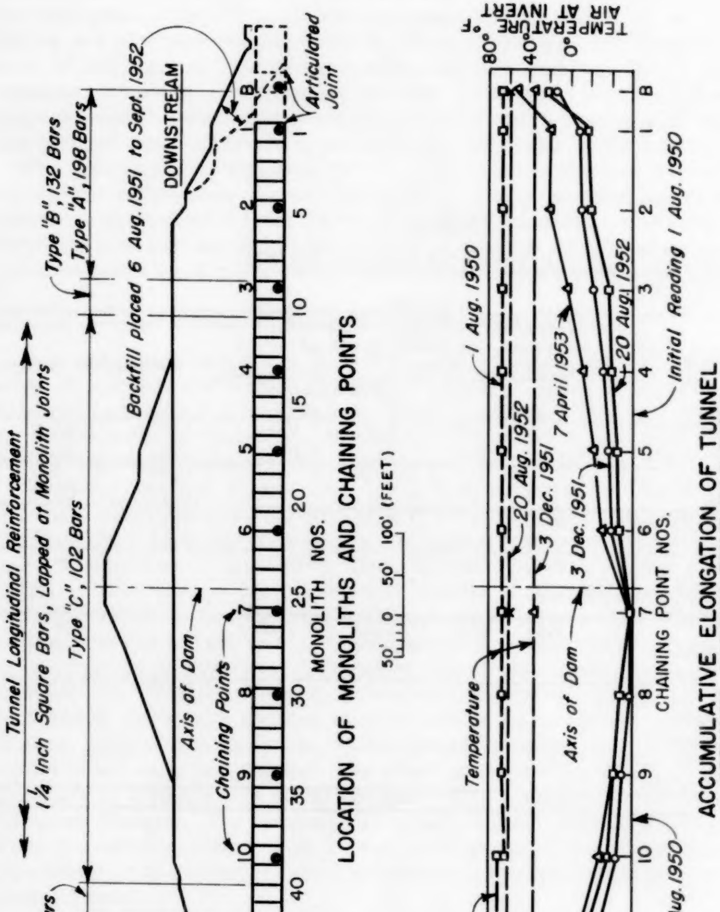
Results of elongation measurements in tunnel 5 are shown in Fig. 19 based

TABLE 4

DIAMETER CHANGES OF CONCRETE LININGS

IN INCHES

LOCATION		Change from Mining Adjacent Tunnel approx. one month after adjacent tunnel mined by				Subsequent Change	
		1st Adjacent Tunnel		2nd Adjacent Tunnel			
Tunnel	Monolith	Horiz. Dia.	Vert. Dia.	Horiz. Dia.	Vert. Dia.	Horiz. Dia.	Vert. Dia.
7	25 (℄ Dam)	-	-	0.07	-	0.09	-
7	18	-	-	-	-	0.13	0.18
5	24 (℄ Dam)	0.14	-	0.14	-	0.10	-
5	34	0.05	0.09	0.10	0.07	0.11	0.10
4	B1 Solid Lining	0.03	0.06	0.06	0.03	-	-
4	D Slotted Lining	0.11	0.13	0.12	0.11	-	-
4	D Solid Lining After Slots Concreted	-	-	-	-	0.054+	0.13+
2	34	0.07	-	0.10	-	0.05	-



ELONGATION OF TUNNEL 5

FIG.19

on the assumption of zero movement at the dam centerline and yielding toward each end. By April 1953 over 2 inches of elongation had been observed with practically no additional elongation after that date. Measurements showed the monolith joint openings were wider in the winter with distinctly less opening near the ends of the tunnel where longitudinal reinforcement was heavier. In the early winter of 1951, the total elongation between portals was 0.09 foot while the sum of the joint opening measurements was about 0.33 foot. This indicates that the joint openings were not primarily due to overall elongation but were attributable mainly to shrinkage and contraction of the concrete due to temperature variation.

SUMMARY

Economy

As covered more fully in the companion paper by Mr. Lane the test section investigation allowed a considerable reduction in the steel rib support for the main tunnels. A major but undetermined saving in construction costs was made possible by having a full size section of tunnel available for inspection and observation by bidders on the main tunnel contract and the construction experience gained in mining allowed the main tunnel contractor to plan operations with considerably more knowledge of actual mining conditions than would have been possible with a smaller pilot tunnel.

Construction

Several inovations in construction, notably the use of jumbos mounted on rails well above the invert and the modification of the mucker to allow continuous loading of muck cars resulted in considerable economy of construction and allowed the project to proceed at a rapid rate. The Fort Union material turned out to be much more favorable for mining than was originally anticipated allowing a mining sequence of two ribs per blast. Little difficulty was encountered with seepage and the blocky Fort Union did not exhibit significant swelling nor squeezing properties. Full length mining was feasible before concreting was started in each tunnel, thus greatly simplifying the construction procedure. The yielding rib section indicated a sizeable decrease in pressure could be obtained but the pressures on the steel rib sections were sufficiently low that such special precautions were not considered necessary for the main tunnels. The excavated bore was well filled by the concrete lining so only a minimum of grouting was required in the crown.

Instrumentation

The mechanical instrumentation performed far better than the electrical instruments. The Whittemore strain gages performed very well, as they have in other observation programs, and the extensometer tapes were quite successful. While the trends established by the pressure cells apparently reflect the load changes rather well, the pressure cells were not satisfactory for quantitative measurements. Tangential components complicate the loading to such an extent that, with the present state of pressure cell design, they are not considered suitable for use in installations similar to the test tunnel. Carlson strain meters operated satisfactorily throughout the observation period but an established base from which to determine quantitative results

is necessary. The SR-4 gage installation was unsatisfactory.

Measurement of stress in the reinforcing steel after chipping out the surrounding concrete was satisfactory but effects of temperature changes complicate the stress relationships to such an extent that similar observations are recommended only where the temperature is fairly constant.

The temperature changes throughout the investigation program were greater than were originally anticipated. Most of the early measurements were made at relatively constant temperatures, but during later phases of the investigation temperature changes masked load effects to a considerable extent. Should such an investigation be undertaken again, it would be well to make provision for regulation of temperature throughout the entire observation period.

Paper 1439 November, 1957 SM 4

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GARRISON DAM—EVALUATION OF RESULTS FROM TUNNEL TESTS SECTION

K. S. Lane,[1] M. ASCE
(Proc. Paper 1439)

SYNOPSIS

This paper describes the principal methods employed in evaluating the observations from the large (36 foot diameter) tunnel test section at Garrison Dam and the major results therefrom—including load on the temporary support ribs, magnitude and distribution of load on the permanent concrete lining, and a better understanding of the inherent safety of a circular lining provided by ring action. Some new analytical methods are presented and their possible application is discussed.

INTRODUCTION

The problem of loads reaching a tunnel in soil or weak rock is sufficiently complex that it is unlikely its solution will ever be developed on a fully rational basis. It depends not only on ground conditions but also on stiffness of the tunnel section since the stiffer the tunnel the more load it is likely to attract. Thus lessons from experience with other tunnels properly play a major role in tunnel design but are applicable only insofar as the similarity of both tunnel sections and ground conditions can be evaluated for adequate comparison. Where available experience seems inapplicable, an alert but humble "feel-your-way" procedure is advantageous.

Such was the situation at Garrison Dam since there was no experience with tunnels in the local formation—Fort Union clay-shale, which contains beds of lignite coal and can be described either as a heavily preconsolidated

Note: Discussion open until April 1, 1958. Paper 1439 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, Vol. 83, No. SM 4, November, 1957.

1. Chief, Foundations and Materials Branch, Kansas City Dist., Corps of Engrs., Kansas City, Mo.; formerly Chief, Soils and Geology Branch, Garrison Dist., Riverdale, N. Dak.

hard clay or as a soft rock. Here the outlet works involved eight large circular tunnels with bore diameters up to 35 feet. These were placed at a relatively close spacing of about one bore diameter between tunnels to achieve over-all economy. Since the record of experience has frequently shown difficulties when tunneling in stiff clays,[2] and since early laboratory tests gave some indication of an unpleasant similarity between the Fort Union and the energetically swelling clays of Paris and Belgium, it was decided to construct a full size 240-foot length of one of the main tunnels in the early stages and equip this as a test tunnel to derive information for guiding the design and construction.

The conduct of this test tunnel investigation, its principal results and the construction methods employed for the main tunnels have been described in the accompanying paper by Mr. Burke. In this paper the most significant results are discussed from the standpoint of their evaluation and application, both as utilized for the Garrison tunnels and as might be considered for future tunnels.

Tunnel Loads

Loading Cases

Fig. 1 shows contours of total overburden load after placing the embankment of the dam and also illustrates the tunnel layout including location of the test tunnel in tunnel 4, and the sections in other tunnels where stresses were measured on steel ribs of the temporary support. These load contours were determined by dividing the embankment loading and the excavation unloadings into a series of strip loads and then computing the net stress at crown of the tunnels for the Boussinesq case of stress distribution. The test tunnel was constructed prior to placing the embankment in a region where the initial overburden load was about 6.6 tons per square foot (t.s.f.). Before mining the main tunnels the embankment was added in order to avoid a load change after tunneling. As shown by Fig. 2, this resulted in increasing the load on the test tunnel to 6.8 t.s.f. in section 4A and to 8.3 t.s.f. in section 4D. Then the main tunnels were constructed in the order shown progressively in Fig. 3a to 3d which created the following general load cases:

Single Tunnel Case.—A single tunnel mined and temporarily supported by steel ribs and then converted to the permanent concrete lining. This applied to the test tunnel and tunnels 2, 5 and 7 for the period prior to mining the adjacent tunnels on each side.

Surface Loading Case.—Addition of a load at the ground surface, in this case the embankment. This applied only to the test tunnel, since for other tunnels the embankment was constructed prior to mining.

Multiple Tunnel Case.—Due to adjacent mining on each side of a previously built tunnel, whence the load is shared between three tunnels and the intervening ground pillars. This applied to the test tunnel and tunnels 2, 5 and 7.

While several other cases are indicated by Fig. 3, such as mining between two previously concreted tunnels or beside one of them, the observations did

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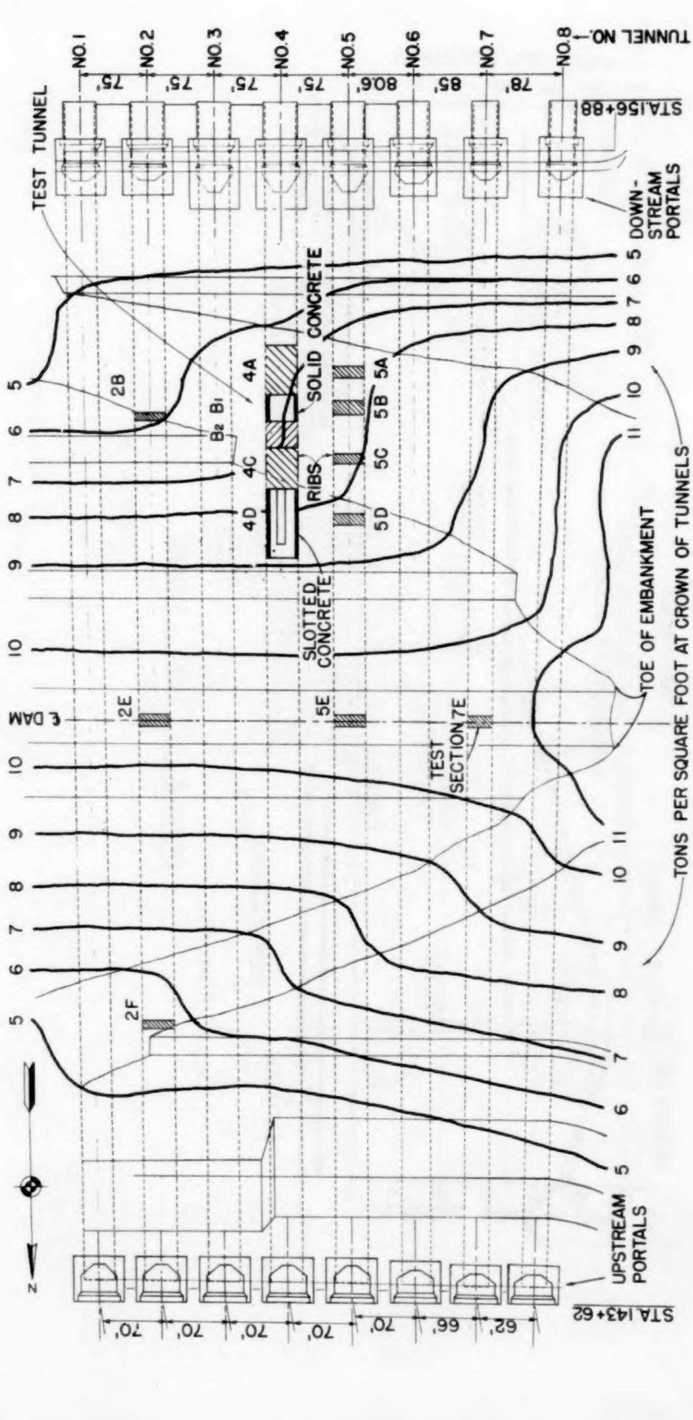
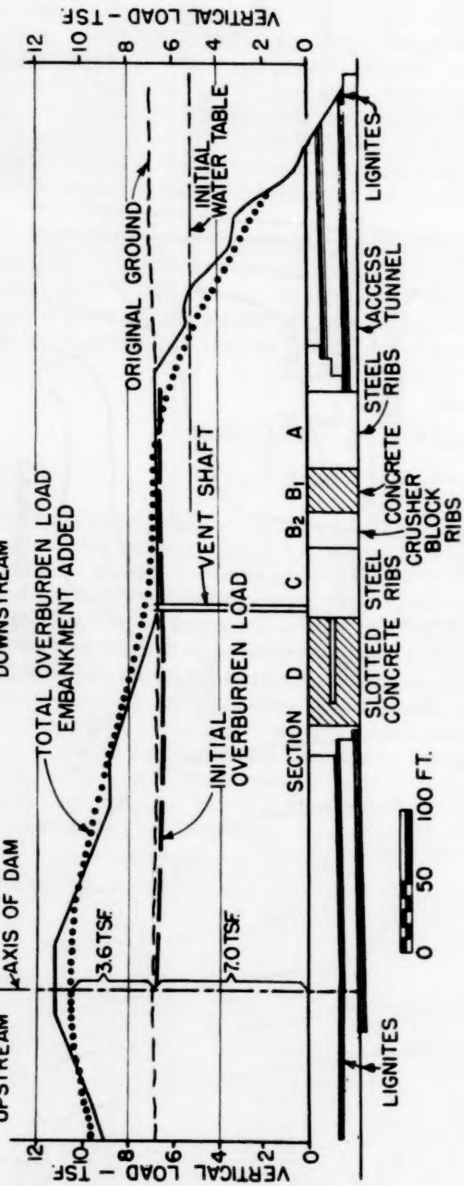
2. For fascinating accounts of early tunnels see:
   2a. "Tunneling" by H. S. Drinker, John Wiley, 1888 and
   2b. "Practical Tunneling" by F. W. Simms, 4th Ed., London, 1896.



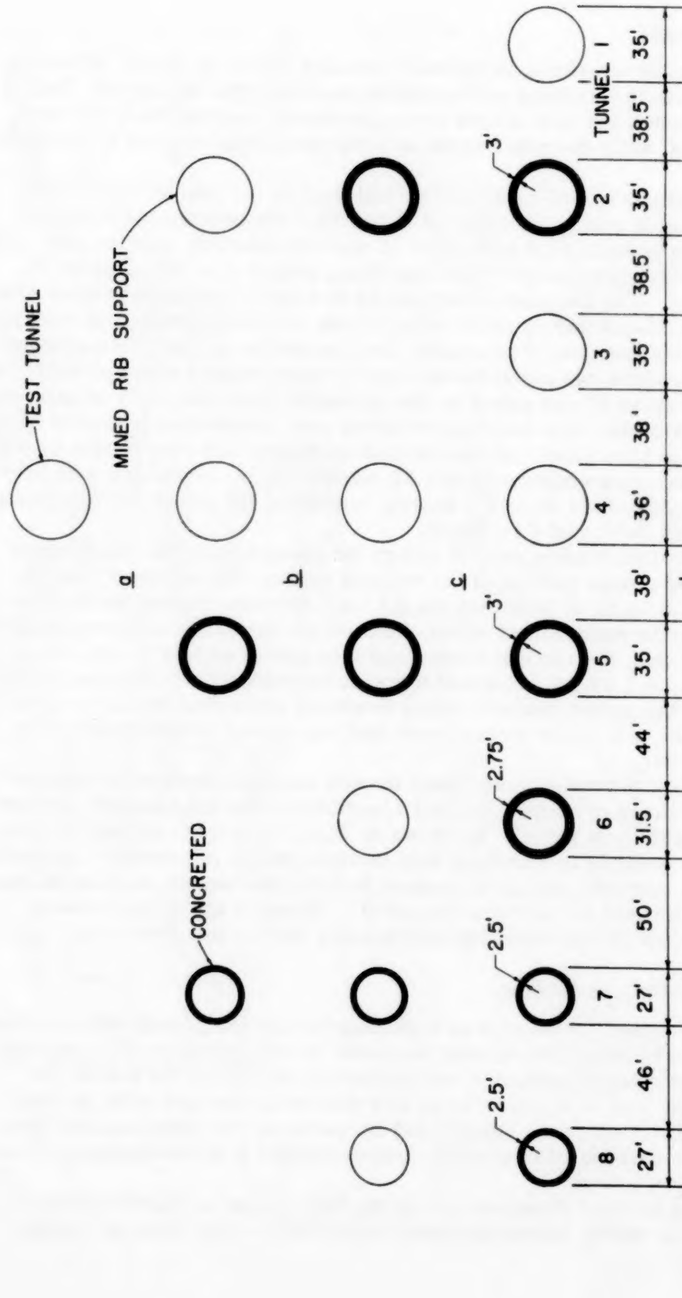
TOTAL OVERBURDEN LOAD

FIG. 1



PROFILE OF TEST TUNNEL

FIG. 2



ORDER OF CONSTRUCTION
OPPOSITE TEST TUNNEL
FIG. 3

not cover such cases except to show them as less severe than the above three cases.

Measured Loads

Typical loads measured on the steel ribs are shown by Fig. 4, while Fig. 5 shows the loads measured on the slotted concrete test section 4D. Except in the test tunnel, the data covers the single tunnel case for the 2-1/2 to 4 months period while the ribs served as temporary support prior to concreting.

For this single tunnel case, the vertical load on the ribs of the 35 to 36-foot bore tunnels was in the range of 10 to 15% of the overburden load—see Table 1. The vertical load on section 7E was considerably smaller, due probably to a greater percentage of the load being arched over this smaller diameter tunnel. The low load on section 4B was due to the much greater yielding resulting from crushing of the wood blocks which were placed at each rib joint in this test section. The smaller load on section 4C, as contrasted with 4A, was considered due partly to the slightly more flexible ribs and different mining methods in 4C and partly to the arching of some load over to adjacent concrete section 4D—this latter point having been independently studied by Sabry.[3] It is significant that the rib load increased with overburden load as shown by comparing sections 5A and 5B outside the toe of the dam with section 5E at center of the dam—the loading conditions for tunnel 5 being similar to those shown for tunnel 4 in Fig. 2.

For the surface loading case of adding the embankment, the contribution from this load change increased the vertical load on rib section 4C only by about 0.1 - 0.2 t.s.f., or less than the 0.5 t.s.f. increase in overburden load computed for the embankment effect. In contrast the stiffer concrete section 4D attracted load, the vertical tunnel load increase of around 2 t.s.f. being greater than the 1.3 t.s.f. computed increase in overburden. Because of this tendency for the stiffer concrete lining to attract more than its share of an added load, the bulk of the embankment load was placed before constructing the main tunnels.

For the multiple tunnel case, while limited measurements were taken on the concrete lining in tunnels 2, 5 and 7, sufficient data for analysis were obtained only in the test tunnel. As shown by Figs. 4 and 5 the mining of an adjacent tunnel resulted in a definite load increase on the previously constructed tunnel. This increase was much greater on the stiff concrete section 4D than on the more flexible rib sections 4A and 4C. At end of the measurements concrete section 4D was carrying substantially 100% of the overburden load.

Extent of Doming and Arching

A recently mined tunnel acts as a yielding hole in the ground with a portion of the load above being arched over the tunnel to the ground at each side and with the balance being carried by the temporary support of the tunnel. At the heading the load is transferred in four directions (to each side, forward to the ground in front of the heading and backward to the tunnel support) as a doming action which is of interest in considering the possible transfer of load

3. "Analysis of Field Observations on the Test Tunnel at Garrison Dam," by Aly A. A. Sabry, Doctorate thesis submitted to Univ. Illinois, Urbana, Ill., 1952.

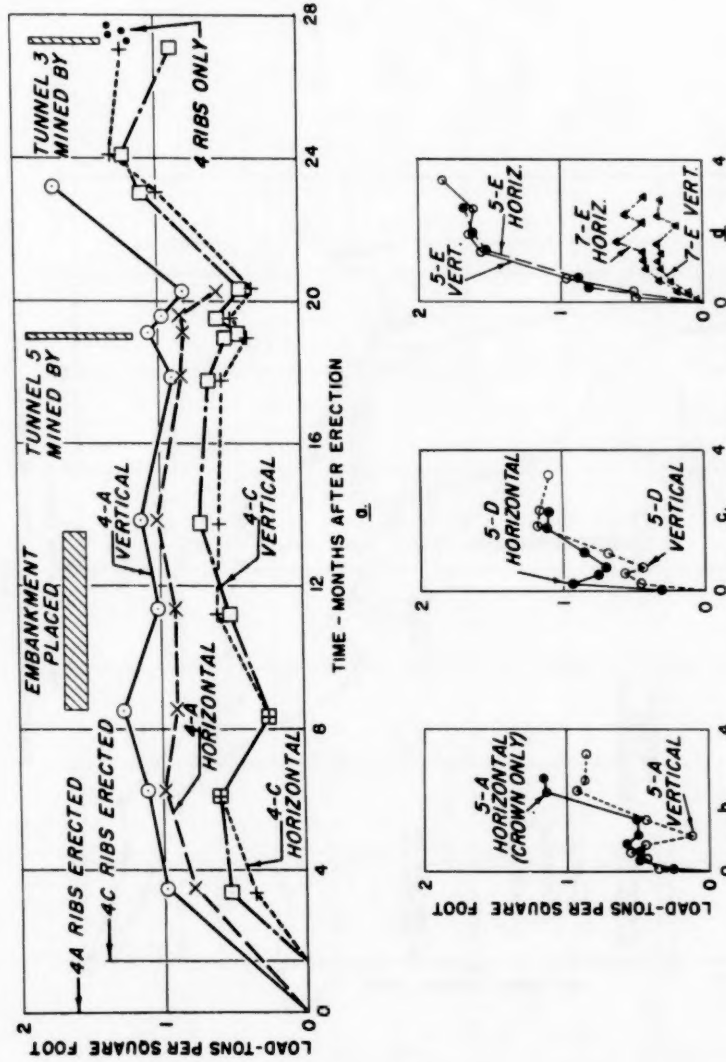
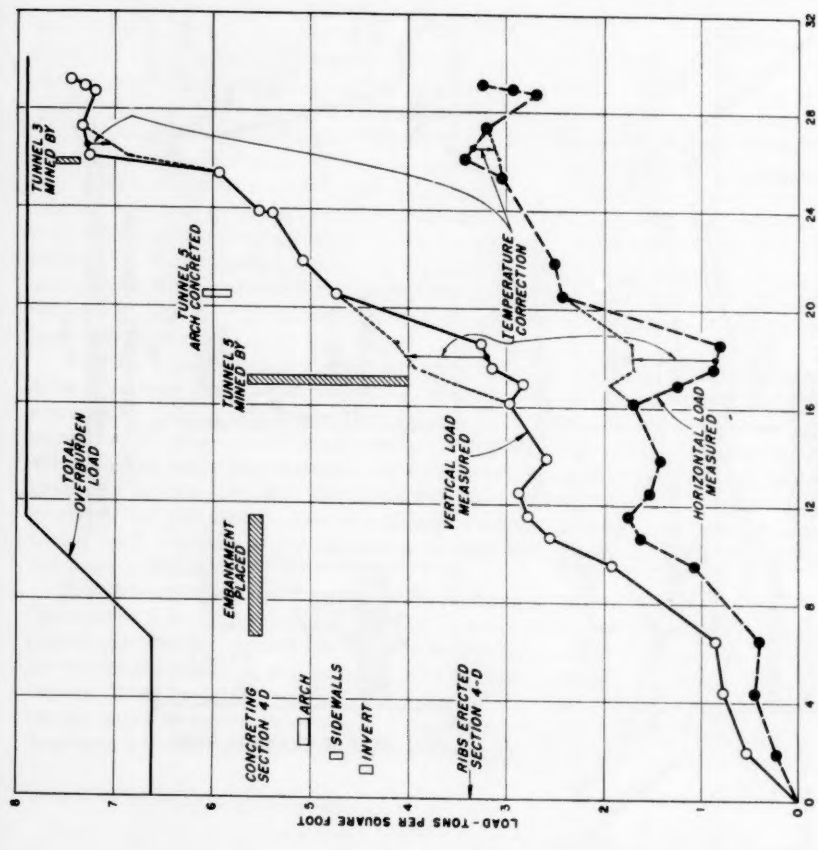


Fig. 4

TYPICAL MEASURED LOADS ON RIBS



MEASURED LOADS-SLOTTED CONCRETE SECTION 4D

Fig.5

TABLE I

Vertical Loads Measured on Ribs

Single Tunnel Case

Tunnel	Section	Time	Overburden Load	Vertical Load	
				Total	% Overburden
36 Ft. Bore					
4	A	Nov. 1948	6.6 tsf	1.0 tsf	15.1%
	B	before add.	6.6	0.3	4.5
	C	embk.	6.6	0.6	9.1
35 Ft. Bore					
5	A	May 1950	7.3	0.85	11.6
	B	do	7.7	0.8	10.3
	C	do	7.8	1.05	13.7
	D	do	8.3	1.15	15.1
	E	do	10.7	1.6	14.9
2	B	July 1950	5.9	0.6	10.1
	F	do	5.7	0.45	7.9
	E	do	10.8	1.55	14.6
27 Ft. Bore					
7	E	March 1950	10.9	0.35	3.2

TABLE 3

RETROSPECT LOAD COMPUTATION

Tunnels 2, 5, & 7				Test Tunnel			
Sec.	Over-burden Load	Computed by Rel. Yield	Meas. Load	Sec.	Over-burden Load	Computed by Rel. Yield	Meas. Load
Single Tunnel Case - Before embankment							
							Nov. '48
				4A	6.6	1.1	1.0
				4C	6.6	0.55	0.6
				4D	6.6	0.7 Ribs alone	0.55
Surface Loading Case - Embankment added							
							Sept. '49
5A	7.3	(7.3)	On soil block before mine tunnel	4A	6.8	(1.15)	1.1
5C	7.6	(7.9)		4C	7.1	(0.6)	0.7
5D	8.3	(8.2)		4D	7.9	(3.2)	3.4 2.8*
Multiple Tunnel Case - After embankment							
			June '50				July '50
5A	7.3	(0.55)	0.85	4A	6.8	(1.55)	1.7
5C	7.8	(0.6)	1.05	4C	7.1	(0.8)	1.1
5D	8.3	(0.5)	1.15	4D	7.9	(6.0)	5.5 4.7*
Single Tunnel Case - After embankment							
5E	10.7	1.8	1.6				
7E	10.9	1.4	0.35				
2E	10.8	1.4	1.55				
2B	5.9	0.5	0.6				
2F	5.7	0.5	0.45				

All loads vertical in tons per square foot
Values in ( ) for n = 2; otherwise for n = 1
* From web and flange stresses; otherwise from web stresses only.

between adjacent tunnel sections of different rigidity. From a construction standpoint it is also significant as indicative of the maximum distance that can be mined ahead before erecting support. As here used, the term doming is applied to the load transfer at the heading and the term arching to the load transfer to each side of the tunnel behind the region of doming, which of course advances with progress of the heading.

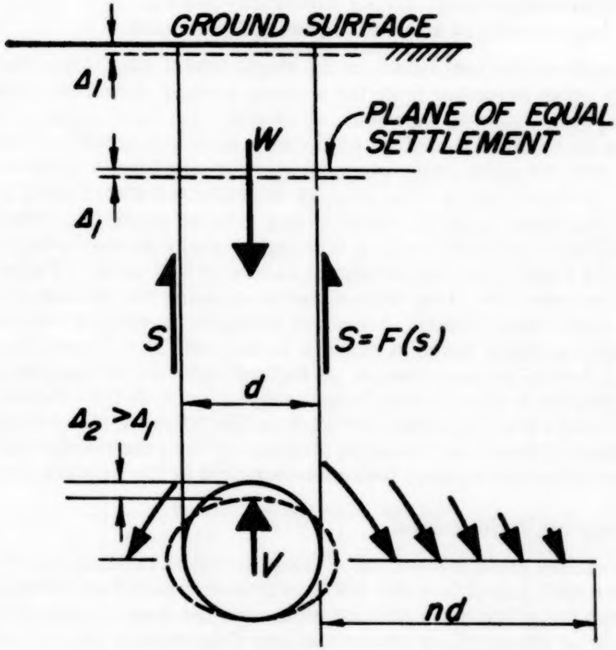
For the single tunnel case some evidence of the lateral extent of doming was furnished by piezometers installed from the vent shaft. When the test tunnel heading approached to within a distance of about 30 feet, these piezometers showed a definite increase in the pore water pressure which continued as the heading passed. This indicated the area of stress increase as extending at least 30 feet forward of the heading. Further evidence was obtained from plots of diameter change vs. distance from the measuring rib to the heading which generally showed a high rate of diameter change while the heading was advancing to a certain distance beyond the rib and then a decrease in rate as the heading progressed further. This distance to the heading was considered a measure of the extent of doming of the load to the ribs back of the heading and most frequently ranged from 30 to 45 feet with an average of around 38 feet from the data obtained during mining of the test tunnel.

For the multiple tunnel case, a more positive measure of the extent of doming was obtained from the diameter changes in the test tunnel which occurred during the mining of tunnel 5 alongside—see Fig. 14 of the accompanying paper by Mr. Burke. When the heading of tunnel 5 approached to about 60 to 80 feet from any rib measured in tunnel 4, its diameter change began to accelerate, indicating that the effect of doming extended about this distance forward of the heading of tunnel 5. As this heading passed the measuring rib, the rate of diameter change became a maximum and then gradually decreased as tunnel 5 advanced and changed the load transfer condition from that of doming to that of arching. Since the continued yield of the ribs of tunnel 5 was accompanied by comparable yield in the opposite sections of tunnel 4, the arching across tunnel 5 extended at least the 70 foot distance over to tunnel 4, or two bore diameters.

These observations indicate the extent of doming was about one diameter or 35 feet in the single tunnel case and at least twice this in the multiple tunnel case. The evidence also shows the extent of arching was of the same general magnitude. While the rate of mining tunnel 5 opposite the test tunnel (about 15 feet per day) was considerably faster than the mining of the test tunnel (about 3 feet per day), it is likely that the increase in extent of arching in the multiple tunnel case was caused mainly by an increase in shear strength of the ground. In the period of 1-1/2 years between mining the test tunnel and then mining tunnel 5, such a strength increase would be expected from consolidation of the ground around the test tunnel under its loading from the single tunnel case and from the added embankment load.

Load by Arching Method

This idea of the extent of arching as a function of the shear strength of the ground led to computing the tunnel loads by the method of arching which is developed in Appendix A. Fig. 6 shows the arching concept where the load arched is transferred to each side of the tunnel by shear on vertical planes so that the load V on the tunnel represents the balance of the overburden load W. The transferred load S depends on the unit shear strength s for which the



ARCHING CONCEPT

FIG.6

following three conditions were recognized, s being a function of the effective or intergranular stress σ.

$s_1 = F(\sigma_1)$—initial condition with effective soil weight buoyant below the original water table and saturated above.
$s_2 = F(\sigma_2)$—condition after lowering the water table by excavation at the downstream portals which exposed lignite beds at the plane of the tunnels. Soil at saturated weight since the open-jointed lignites act as horizontal drains directing seepage forces downward—see Fig. 2.
$s_3 = F(\sigma_3)$—condition after adding embankment load.

Early loads on the test tunnel in the single tunnel case were reasonably checked by those computed from the arching method, based on the initial shear strength s_1—see Table 2. However, as tunnel 7 and later tunnel 5 were mined, the loads there measured on the ribs were much smaller than those computed with the same shear strength s_1. Attempts were made to estimate the extent of consolidation in increasing the effective stress from σ_1 toward σ_2 and even toward σ_3 which resulted in a value of s near s_2. While the computed loads were reduced by using this higher shear strength, they were still considerably higher than the measured values as indicated in Table 2.

This illustrates one of the difficulties in applying the method of arching where its controlling element, the shear strength, is varying with the progress of consolidation which latter is difficult to estimate at different times over the short period of measurements. A further difficulty is the lack of differentiation between a very flexible tunnel support such as the crusher block test section 4B and a very rigid support such as the concrete test section 4D. Consideration of these variations in stiffness of both the ground and the different types of tunnel support led to development of the relative yield method.

Load by Relative Yield Method

In the relative yield method the arching approach is modified to take cognizance of the well known fact that the load attracted to a rigid structure increases with the stiffness of that structure and the load thrown off or arched over a yielding structure increases with the flexibility of the structure. The basic concept is shown by Fig. 7 and considers a group of elastic blocks in the plane of the tunnels which have different deformation moduli—E_t for a flexible rib support being less than E_s for the ground and E_c for a stiff concrete lining being greater than E_s. The ground above the tunnels is considered as a series of blocks with the loads transferred between adjacent blocks by shears on their boundaries. The lower boundary at the invert is assumed either to be rigid or to settle uniformly so that the significant deflections occur in the height d of the elastic blocks. With these simplifying assumptions the relations for the vertical tunnel loads are derived in Appendix B, the final equations for the three principal load cases being as follows:

<u>Single Tunnel Case</u>.—load on flexible rib support.

$$V = W\left[1 - \frac{n}{b + n}\right] \tag{1}$$

<u>Surface Loading Case</u>.—load increase from adding a unit load, p, at the surface.

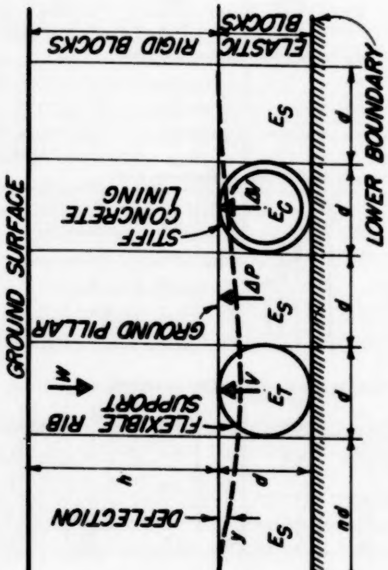
On stiff concrete lining

TABLE 2

INITIAL LOAD COMPUTATION

Tunnels 5 and 7						Test Tunnel					
Sec.	Over-burden Load	Computed Load Arching f (s_1)	 f (s_2)	 Rel. Yield	Meas. Load	Sec.	Over-burden Load	Computed Load Arching f (s_1)	 f (s_2)	 Rel. Yield	Meas. Load
Single Tunnel Case - Before embankment											
											Nov. '48
						4A	6.6	0.9		0.9	1.0
						$4B_1$	6.7	1.0		0.05	0.4
						$4B_2$	6.7	1.0		0.05	0.3
						4C	6.6	0.9		0.4	0.6
						4D	6.6	0.9	Ribs Alone	0.7	0.55
Surface Loading Case - Embankment added											
											Sept. '49
5A	7.3	(8.0)		(8.0)	On soil block before mining Tunnel 5	4A	6.8	1.1		0.9 (0.5)	1.1
5B	7.7	(8.4)		(8.4)		$4B_1$	7.0	1.3		0.05 (0.03)	0.4
						$4B_2$	7.0	1.3		0.05 (0.03)	0.4
5C	7.8	(8.5)		(8.7)		4C	7.1	1.4		0.4 (0.3)	0.7
5D	8.3	(9.0)		(9.2)		4D	7.9	2.2		2.8 (3.0)	3.4 2.8*
Multiple Tunnel Case - After embankment											
					June '50						July '50
5A	7.3	1.1 (1.8)	0 (0)	0.2 (0.1)	0.85	4A	6.8	1.1 (1.9)	1.1 (2.1)	0.9 (0.9)	1.7
5B	7.7	1.5 (1.8)	0 (0.2)	0.3 (0.1)	0.8	$4B_1$	7.0	1.3 (2.1)	1.3 (2.3)		
						$4B_2$	7.0	1.3 (2.1)	1.3 (2.3)		
5C	7.8	1.6 (2.3)	0 (0.3)	0.3 (0.1)	1.05	4C	7.1	1.4 (2.2)	1.4 (2.4)	0.4 (0.5)	1.1
5D	8.3	2.1 (2.8)	0.7 (0.8)	0.3 (0.1)	1.15	4D	7.9	2.2 (3.0)	2.2 (3.2)	2.8 (6.3)	5.5 4.7*
Single Tunnel Case - After embankment											
5E	10.7	3.7	2.5	0.35 (0.15)	1.6						
7E	10.9	2.6	0.6	0.4 (0.1)	0.35						

All loads vertical in tons per square foot
Values in ( ) for n = 2; otherwise for n = 1
* From web and flange stresses; otherwise from web stresses only



RELATIVE YIELD CONCEPT

FIG. 7

$$(\Delta V)\ \text{concrete} = pd\left[1 + \frac{n(c-1)}{c+n}\right] \tag{2}$$

On flexible rib support

$$(\Delta V)\ \text{ribs} = pd\left[1 - \frac{n(1-b)}{b+n}\right] \tag{3}$$

Multiple Tunnel Case.—from mining adjacent to a previously concreted tunnel.

Load on rib support of the tunnel being mined

$$V = W\ \frac{b}{(b+2c+3)} \tag{4}$$

Load increase on the concreted tunnel

$$\Delta V = W\ \frac{c}{(b+2c+3)} \tag{5}$$

Load increase on the intervening ground pillar

$$\Delta P = W\ \frac{1}{(b+2c+3)} \tag{6}$$

These load equations depend on the arching distance nd and on the stiffness ratios (tunnel block to ground block)

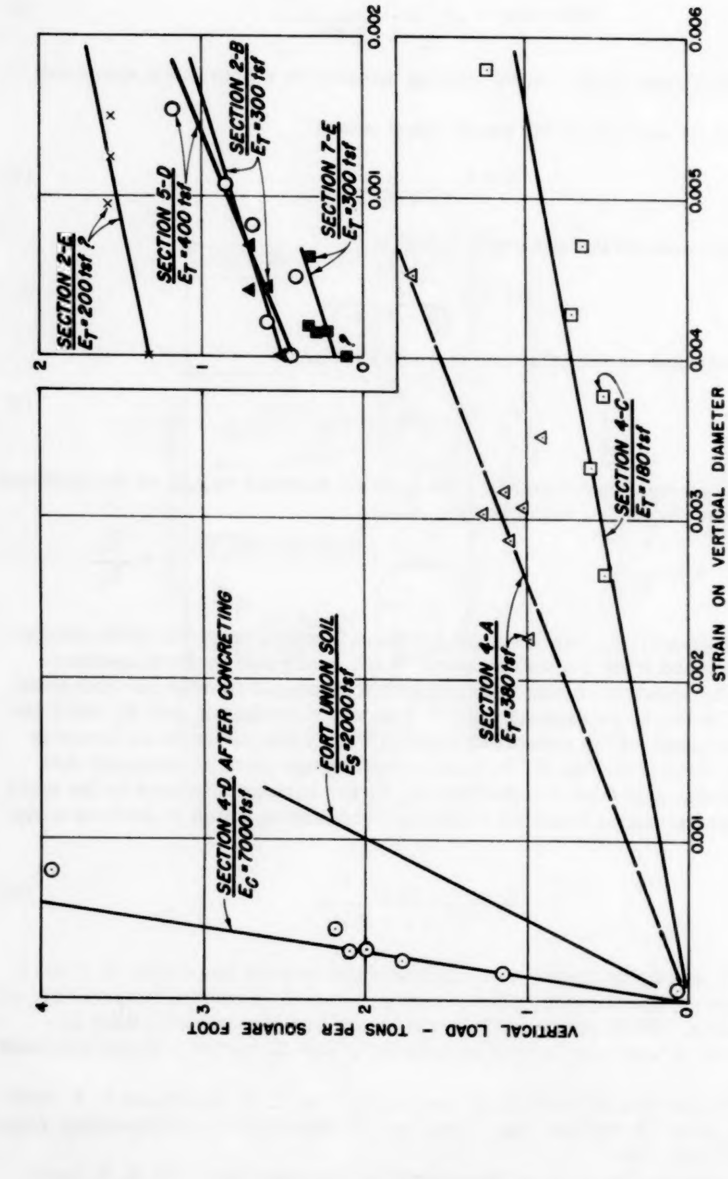
$$b = \frac{E_t}{E_s} \qquad \text{and} \qquad c = \frac{E_c}{E_s}$$

For the ground, E_s was taken as 2,000 t.s.f. which value had been reasonably established from laboratory tests[4] and confirmed by field measurements of the rapid or elastic rebound, which amounted to over one foot from unloading by the large excavations.[5] The tunnel moduli E_t and E_c were obtained from plots of the measured vertical load V vs. the vertical diameter change D_y similar to Fig. 8. In early computations when preliminary data were available only from the test tunnel, E_t for various locations in the main tunnels was estimated from the following relationship which is derived in Appendix C:

$$E_t = \frac{12\ EI}{R^3\ S(1 - H/V)} \tag{7}$$

where H/V is the horizontal-vertical load ratio and the quantities R, S and I are the radius, spacing and cross sectional moment of inertia respectively of the steel ribs. While equation (7) might be applied directly as a first approximation, it was more useful by indicating how E_t varied with its pertinent

4. "Soil Properties of Fort Union Clay Shale," by C. K. Smith and J. F. Redlinger, Proc. Third Internat. Conf. on Soil Mechanics and Foundation Engr., Switzerland, 1953.
5. "Designing for Foundation Movements at Garrison Dam," by K. S. Lane, Proceedings of Fifth Congress on Large Dams, Paris, 1955.



TYPICAL DEFORMATION MODULI

FIG. 8

elements—i.e. directly with I, inversely with R^3 and inversely with S. Hence for initial computations, moduli for the main tunnels were assumed to vary according to these relationships from the value of E_t determined for test section 4C.

Results of these computations with preliminary data are presented in Table 2. In the test tunnel the check with measured loads was good with the load arched 1 diameter (n = 1) for the single tunnel and surface loading cases, while for the multiple tunnel case it was better with the load arched 2 diameters (n = 2). In tunnels 5 and 7 the computed load was consistently lower than that measured but the check was adequate for practical use in estimating the vertical load for selecting rib sizes since, as discussed later, this was controlled more by bending moment than by direct thrust.

On completion of the investigation the computations were repeated using final data from tunnels 2, 5 and 7 and further data from the test tunnel. Values of E_t were determined from plots of vertical load vs. vertical diameter change, except where the vertical diameter was not measured, the horizontal was used and assumed equal to the vertical. Table 3 shows the results of this retrospect computation, based on the load arched 1 diameter in the single tunnel case and arched 2 diameters after this, considering the shear strength of the adjacent ground pillars increased by consolidation under the loading arched over by mining of the single tunnel. In the test tunnel the check of the measured loads was good for all three cases, considering that the load measured in section 4D was so complicated by a temperature change during the mining of tunnel 5 that it was rather difficult to estimate the proper value of the measured load. In the other tunnels the check was similarly good except in tunnel 7 which had the smallest diameter of the tunnels measured.

While closer checks could doubtless be obtained by further refinements (one being an allowance for further yield of the ground pillar due to consolidation which would gradually increase the load on the tunnel), the relative yield method proved adequate for practical usage and checked the measured loads considerably closer than the method of arching. The method, of course, involves uncertainties in choosing values of E_s, n and E_t (common to most problems in choosing soils and materials constants); but in this application the pertinent ratios of these factors did not vary widely as:

n was certainly not less than 1 nor more than 3

$b = \frac{E_t}{E_s}$ for the sizes of ribs used appeared to vary in the range of 0.1 to 0.2 (except for the unusually flexible crusher block test section)

$c = \frac{E_c}{E_s}$ was used in the range of 3 to 4 for the slotted concrete test section and probably was somewhat greater for the full circle concrete lining of the main tunnels.

A major advantage of the relative yield method was its ability to picture the load transfer between tunnels and ground—away from yielding points and toward stiff points. For example, comparing equations (2) and (3), these expressions show how an added surface load is attracted by a stiff concrete tunnel and thrown off by a tunnel with yielding support. From the test tunnel data the small stiffness of the rib support was very apparent (of the order of 1/5 to 1/10 that of the ground), also the great stiffness of the concrete lining —(3-1/2 or more times that of the ground). Not only did the stiffness of the

tunnel lining determine the amount of vertical load reaching the tunnel; it also was the principal factor controlling the amount of horizontal load.

Horizontal—Vertical Load Ratio

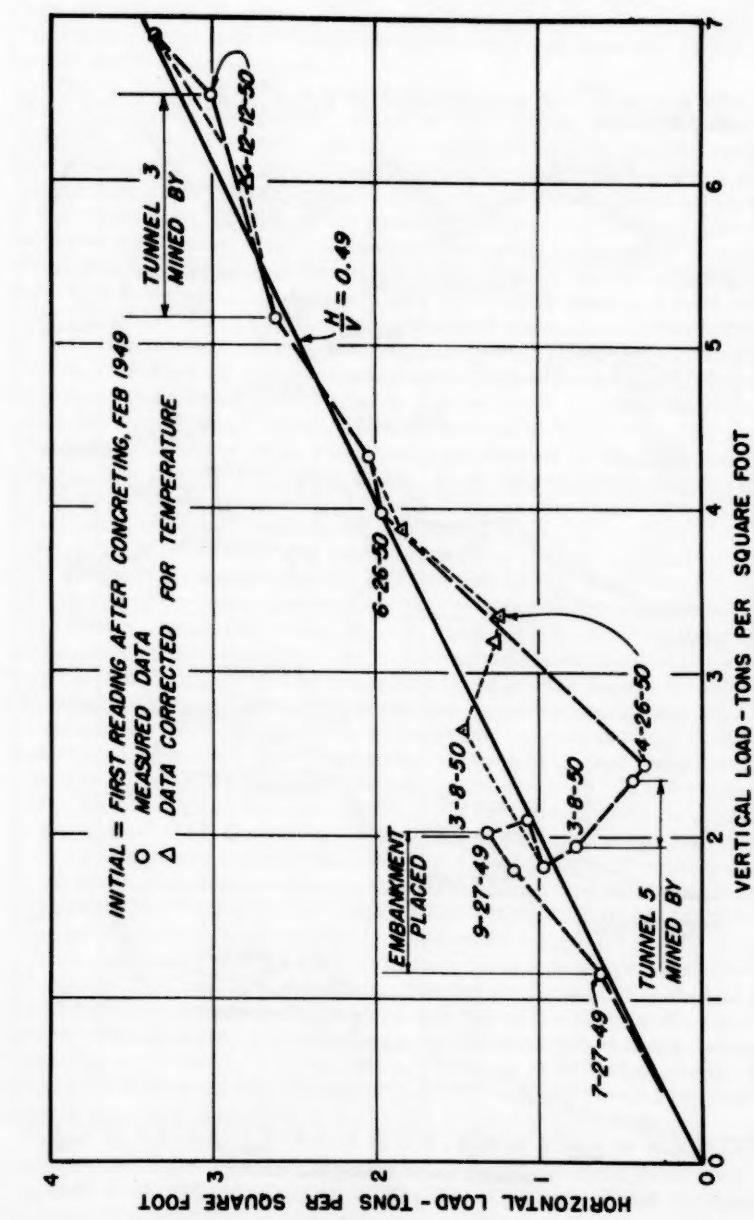
At nearly all of the measuring sections the vertical load was consistently greater than the horizontal; the vertical diameter shortened; and the horizontal diameter increased. This clearly indicated the vertical as the activating load causing the vertical diameter to decrease and the horizontal to lengthen, thus forcing out the support at the spring lines to build up the horizontal as the passive load. This performance is shown by the data on Fig. 4 and also by Fig. 12 of the companion paper by Mr. Burke.

The only significant exceptions were sections 2F and 7E where the horizontal load was measured slightly greater than the vertical and the diameter changes behaved erratically. Since tunnels 2 and 7 were outside tunnels at the time of mining, it is conjectured that residual horizontal stress in the Fort Union could have been responsible for the horizontal load being slightly greater than the vertical. The geologic history of the site is compatible with the presence of such residual horizontal stresses since at one time the area was loaded by around 1,000 feet of additional overburden which would have caused comparatively high horizontal stresses. Subsequently this overburden was eroded, followed by cutting of the river valley which latter would allow a lateral yield to reduce the horizontal stress. Some evidence of residual stresses was observed at base of the powerhouse excavation where vertical slots tended to close a few hours after cutting by a coal saw; however, this occurred near the bottom of the pre-glacial river valley and was not observed at the higher elevation of the tunnels.

In the single tunnel case the horizontal load on the flexible rib support was approximately equal to the vertical—the horizontal-vertical load ratio ranging from 0.8 to 1.1. In the stiff slotted concrete section the ratio was much lower, ranging from 0.4 to 0.6 and averaging 0.5 as shown by Fig. 9. This H/V ratio was temporarily reduced by the adjacent mining of tunnel 5, as in rib section 4A it dropped from 0.9 to 0.8 and in section 4C from 0.85 to 0.7, followed in both cases by recovery to its earlier value. In the stiff concrete section 4D this temporary reduction was much greater with the H/V ratio falling as low as 0.25 although the true picture was considerably obscured by the effect of a concurrent temperature drop. It is likely that the horizontal load was temporarily reduced as a result of a small yield laterally toward the open bore of tunnel 5 which was definitely indicated by a temporary drop registered by the pressure cells at the spring line as the heading of tunnel 5 passed by section 4D.

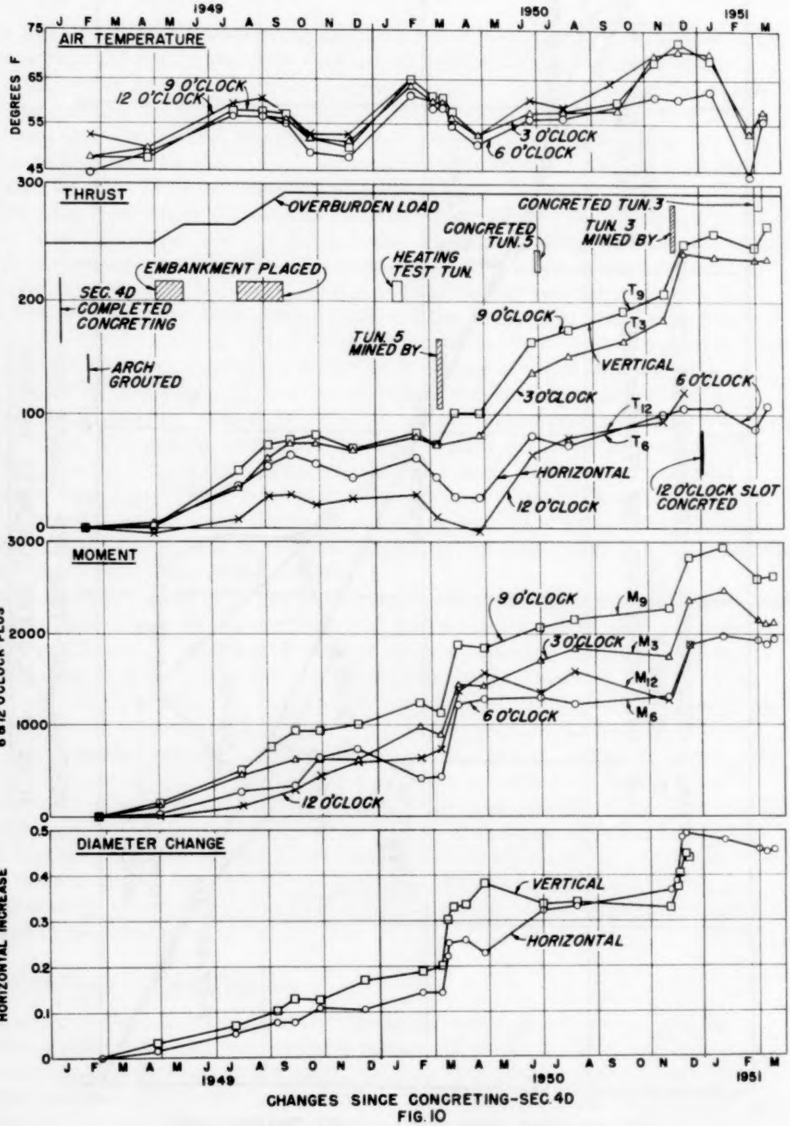
Effect of Temperature Changes

In early stages the temperature inside the test tunnel approached the narrow range of ground temperature (about 50° to 55° F.) and this was also true during mining each of the main tunnels. Thereafter, the temperature varied considerably in the range of 0° to 70° F. as wind blew through the open tunnels or as they were heated when concreting was performed in the winter. During the measuring period the inside temperature in concrete test section 4D varied from 45° to 75° F. which considerably influenced the measured results. This is apparent from Fig. 10 where the data are plotted as changes since concreting, based on the concept that stress in the concrete is derived



H VS. V SLOTTED CONCRETE SECTION

Fig. 9



CHANGES SINCE CONCRETING-SEC. 4D

FIG. 10

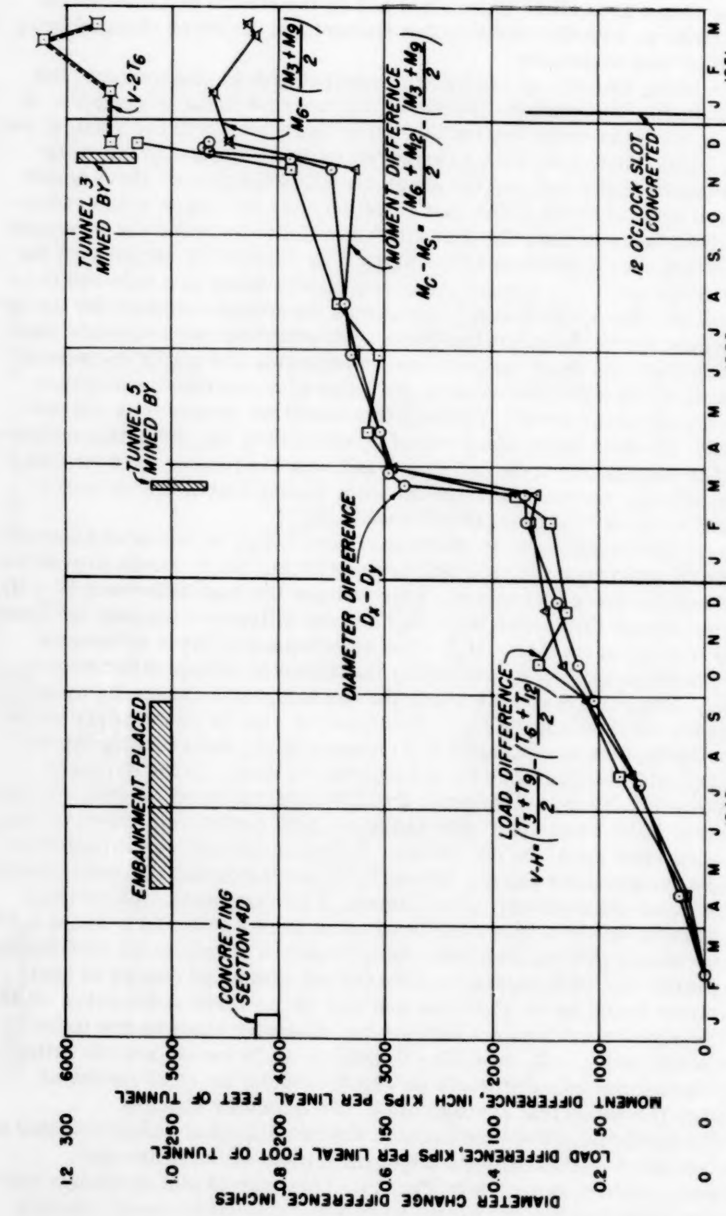
from subsequent load changes. As tunnel 5 was mined adjacent, the expected increase in thrust in section 4D was masked by the effects of a concurrent temperature drop, with the effect on the moment and diameter changes being similar but of less magnitude.

With the lining exterior at essentially constant ground temperature, the effect of an inside temperature change on the concrete lining is complex. A temperature increase would expand the lining increasing both the vertical and horizontal loads. However, with a temperature decrease, the lining would contract away from the soil and the shear stress in the ground above would be increased causing more of the load to be arched over which would reduce the tunnel load, at least until the soil yielded sufficiently under the increased stress to follow the contraction of the lining. By drastically simplifying the conditions to the case of a homogeneous ring tightly fitted in a hole within an elastic medium, Mr. C. K. Smith[6] obtained a theoretical solution for the effect of a temperature change on the thrust. By selecting two relatively short periods with about the same temperature at beginning and end of the period, the measured loads were corrected to the basis of a constant temperature within the chosen short period, although this would not necessarily correct them to their absolute value which would be affected by the prior temperature history. The results are shown on Fig. 5 and with the temperature influence thus approximately removed, the effects from mining tunnel 5 were more nearly equal to those caused by mining tunnel 3.

As a more direct approach to the temperature effect in terms of observed data, a type of difference function plot devised by Mr. H. H. Burke proved the most useful of the many plots tried. This utilized the load difference (V - H), the diameter change difference (D_x - D_y) and the difference between the crown and spring line moments (M_c - M_s). The significance of these difference functions can be visualized by examining the diameter change difference as an example. The normal condition was for the horizontal change D_x to increase and the vertical change D_y to decrease. A rise in temperature would expand the lining, thus accentuating the increase in D_x and reducing the decrease in D_y, with the converse for a temperature drop. In the diameter change plot on Fig. 10 this divergence from the normal trend is apparent during September 1949, December 1949 and April 1950 and is well shown by Fig. 13 of the companion paper by Mr. Burke. Assuming the entire ring affected by a uniform temperature change, the vertical and horizontal diameter changes would be about equal numerically. Hence, if the horizontal and vertical diameter changes due to a particular load event were a plus and a minus 0.63 inches respectively and the diameter change due to a temperature rise during the same period was +0.2 expansion, then the net measured change of horizontal diameter would be +0.83 inches and that of the vertical diameter -0.43 inches. The algebraic difference between the diameter changes due to the load event would be D_x - D_y or 0.63 - (-0.63) or +1.26 inches and the difference in the measured changes would be +0.83 - (-0.43) or +1.26 inches or equal to twice the numerical average of the two diameter changes.

Since this algebraic difference between the two diameter changes tended to eliminate the effect of the temperature change, plots of the difference functions were utilized as shown by Fig. 11. This type of plot strikingly indicated that the principal loading events were accompanied by major changes

6. Formerly Head, Soils Design Section, Garrison District, Corps of Engineers.



DIFFERENCE FUNCTIONS—SLOTTED CONCRETE SECTION

FIG.11

with a gradual change occurring during the intervening periods. In the sign convention used D_y and M_s are negative so that algebraically $(D_x - D_y)$ is twice the average diameter change and $(M_c - M_s)$ is twice the average of the four moments at the crown, invert and both spring lines. The fact that these three difference functions could be matched by adjusting the scales in Fig. 11 shows that they were inter-related. As an example, Fig. 12 shows the moment difference $(M_c - M_s)$ was a linear function of the load difference (V - H).

Bending Moments

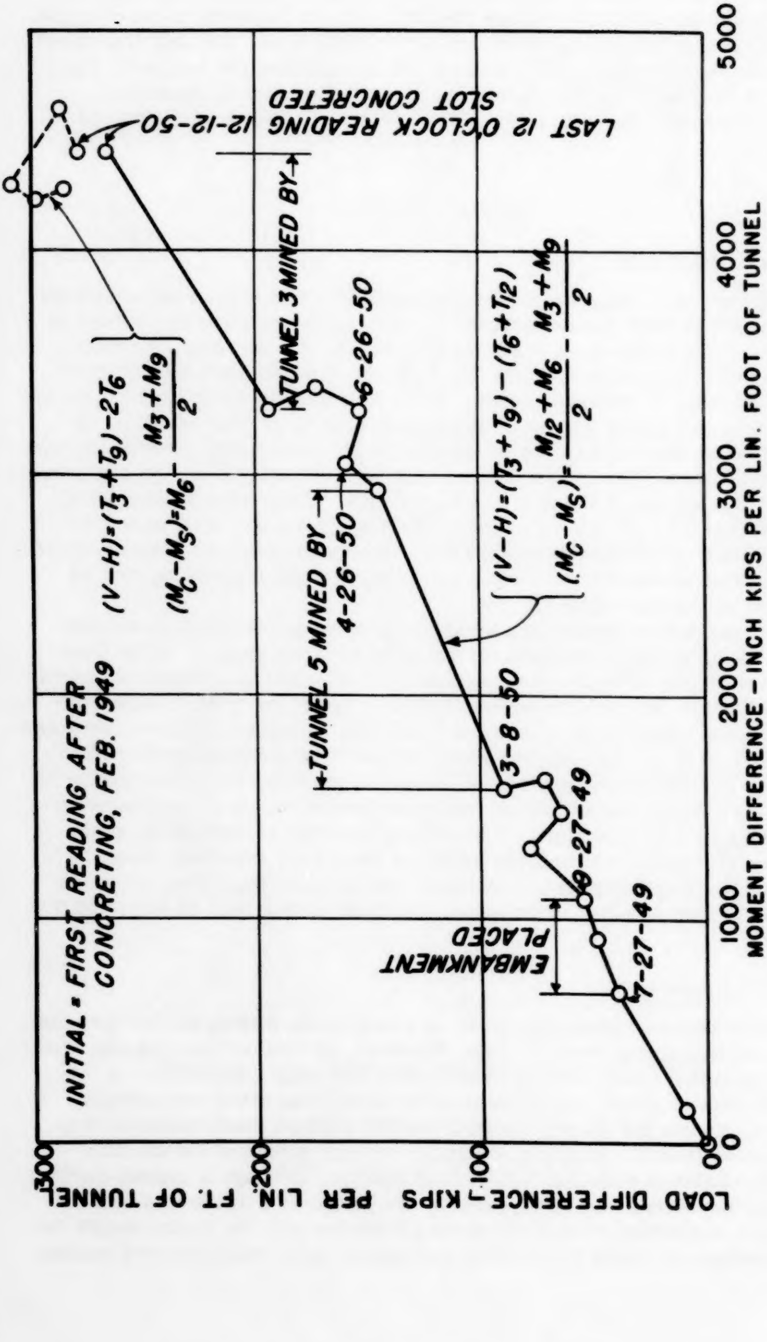
Moments in Steel Ribs

Results of rib measurements for the period the ribs served as temporary support prior to their encasement in the concrete lining are summarized in Table 3 of the accompanying paper by Mr. Burke. All sections represent a single tunnel case except sections 5A, B, C and D which were mined beside the correspondingly lettered sections of the test tunnel. Some of the moments at the invert and spring line were opposite in sign from what would be obtained with the vertical as the activating load and sometimes reversed in sign at different times. The crown moment was most consistent in sign and nearly always the largest of the four clock positions. Comparing section 5A at toe of the dam with section 5E at the centerline, there was no apparent increase in moment as the overburden load increased. Of much greater significance was the increase in moment with stiffness of the supporting ring as illustrated by the curves of Fig. 13.

The stress due to flexure was a high percentage of the total rib stress, around 75%, or about three times the contribution from thrust. Outer fiber stresses of 30,000 to 40,000 p.s.i. occurred at all measuring sections except 4A and 4C where the rib section was unusually heavy for testing purposes—these stresses being either measured directly at the inner flange or computed on the basis of linear distribution using two or three measurements on the web. On a few ribs in the test tunnel, stresses were investigated at some 20 to 80 points closely spaced around the cross section which showed considerable bending in both directions. Considering that this crossbending would result in significantly higher fiber stress at the flange ends than shown by Table 3 of Mr. Burke's paper, it is likely that the maximum fiber stresses on numerous ribs equalled or exceeded the yield point (about 33,000 to 38,000 p.s.i.).

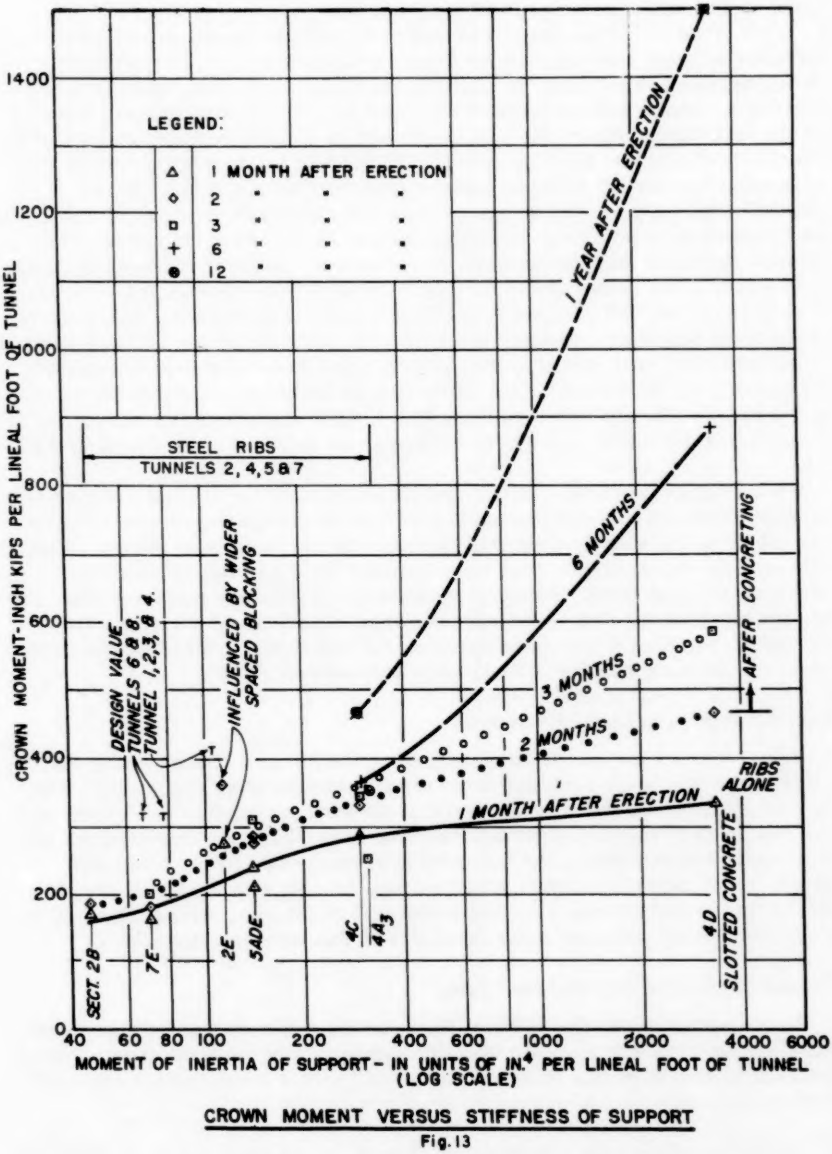
Causes of Moment in Ribs

The most obvious cause of moment in a ring is the difference between the vertical and horizontal loads (V - H). However, on most of the ribs the vertical and horizontal loads were so nearly equal that only a small part of the observed moment could be attributed to the small load difference actually measured. Hence, the major causes of the rib moment were considered as irregularities from the blocking loadings, erection stresses and similar factors inevitable in practical tunnel construction. Perhaps a sizable portion of the erection stress was contributed by the jumbo load which was carried by brackets supporting rails at the spring line—not only the jumbo weight but also the temporary loads from crown and breast jacks which reacted against



(M_C-M_S) vs $(V-H)$ — SLOTTED CONCRETE SECTION

FIG. 12



CROWN MOMENT VERSUS STIFFNESS OF SUPPORT

Fig. 13

the jumbo. However, it is likely that the inevitable irregularity in blocking loads was a more important factor, resulting from variations in the size and spacing of the blocks and in the concentrated load created at each blocking point by the initial and/or subsequent wedging.

R. V. Proctor(7) has suggested that in rock tunneling the rib moment increases with the spacing between blocks because the rise of arc between blocking points increases. In general, the range of blocking spacing in the Garrison tunnels was not sufficiently great to afford a check of this concept. In the test tunnel, where the ribs were used as elastic measuring rings, substantially continuous blocking was placed between the lags which resulted in a reasonably uniform blocking pattern covering perhaps 75% of the rib area available for support. In the main tunnel the ribs were required to be blocked at 16 points as a minimum, including each of the 4 joints; but actual blocking usually exceeded this, particularly in the crown. In general the spacing was 2 to 3 feet in the crown and in the invert about 4 feet, considering both the soil contacts and the intermediate blocks as points of support. At the spring lines there was some tendency for wider and more irregular blocking spacing which probably contributed to the varying signs of moments observed here. Comparing the moments for the close-spaced blocking in the test tunnel with those for the slightly wider blocking in the other tunnels, there was no significant difference when the effect of stiffness was considered as shown by Fig. 13.

Occasionally blocking was lost and loosened when the ground softened from seepage, and one of these instances occurred at measuring section 2E where the effective blocking spacing was thus increased up to about 15 feet. With this spacing the moments were very high and the maximum fiber stress reached the yield point. Hence, it appears for the Garrison tunnels that varying the blocking spacing in the narrow range of perhaps one to three feet caused no significant change in moment, but that a major increase did occur when the blocking spacing inadvertently approached 15 feet.

Moment in Slotted Concrete Section

Fig. 10 shows the moments measured on the 30-inch ribs spanning the slots in section 4D as resulting from stress changes after concreting. The highest moment consistently occurred at the west spring line (9 o'clock) and the moment increased considerably at time of the different load changes with a gradual increase during the intervening periods—this latter being well shown by the difference function plot of Fig. 11. As with the steel ribs, the stress due to flexure was a considerable part of the total stress, about 30 to 45% at the spring lines and about 60 to 75% at the crown and invert.

Causes of Moment in Concrete Lining

Since the horizontal and vertical loads on the concrete test section were quite unequal, the effect of this load difference will be explored first. For a theoretical ring of radius R, the formulas of Table 4 show that for any load distribution

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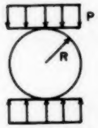
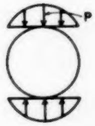
7. "Rock Tunneling with Steel Supports" by R. V. Proctor and T. L. White, P. 207; Publ. by Commercial Shearing and Stamping Company, Youngstown, Ohio, 1946.

TABLE 4
RING COEFFICIENTS

LOAD	RING COEFFICIENTS			
	Mc	Ms	Dx	Dy
UNIFORM 	+0.2500	-0.2500	+0.1667	-0.1667
PARABOLIC 	+0.2188	-0.1979	+0.137	-0.142
DRUCKER 	+0.2075	-0.1816	+0.1277	-0.1310
180° TRIANGLE 	+0.1793	-0.1541	+0.1086	-0.1118
120° TRIANGLE 	+0.1684	-0.1396	+0.0996	-0.1033
90° TRIANGLE 	+0.1512	-0.1189	+0.0862	-0.0904
60° TRIANGLE 	+0.1208	-0.0875	+0.0646	-0.0688

M = Coeff. pR^2

Mc = crown & invert moment
Ms = springline moment

D = Coeff. $\frac{pR^4}{EI}$

Dx = change in horizontal diameter — increase

Dy = change in vertical diameter — decrease

$$M = C\,p\,R^2$$

where p is the unit load and C a constant depending on the load distribution. With V and H the unit vertical and horizontal loads, the crown and invert moment M_c and the springline moment M_s are then

$$M_c = C_1\,R^2\,V - K_2\,R^2\,H \tag{8}$$

$$M_s = K_1\,R^2\,H - C_2\,R^2\,V \tag{9}$$

where C_1 and C_2 depend on the distribution of the vertical load and K_1 and K_2 on that of the horizontal load.

For the general case with V and H unequal and their distributions also different, $(M_c - M_s)$ is a function of $(V - H)$ and of the distribution constants C_1, C_2, K_1, K_2

$$M_c - M_s = R^2\left[(C_1 + C_2)\,V - (K_1 + K_2)\,H\right] \tag{10}$$

For a more specific case if V is unequal to H but both have the same type of distribution curve

$$C_1 = K_1 \quad \text{and} \quad C_2 = K_2 \qquad \text{so} \qquad C_1 + C_2 = K_1 + K_2$$

and $(M_c - M_s)$ is then a linear function of $(V - H)$

$$M_c - M_s = R^2\,(K_1 + K_2)\,(V - H) \tag{11}$$

and by similar derivation the diameter change difference $(D_x - D_y)$ is also a linear function of $(V - H)$

$$(D_x - D_y) = \frac{R^4}{EI}\,(B_1 + B_2)\,(V - H) \tag{12}$$

where the constants B_1 and B_2 depend on the load distribution.

For only the simple distribution of a uniform load does $K_1 = K_2$ so that

$$M_c - M_s = 2K\,R^2\,(V - H) \tag{13}$$

and

$$M_c = -M_s = K\,R^2\,(V - H) \tag{14}$$

For other load distributions K_1 and K_2 are not equal, although they are often nearly so as shown by Table 4.

For any combination of load distributions, equation (10) shows $(V - H)$ as the principal factor responsible for the moment. For the specific case where V and H have the same shaped distribution curve, equations (11) and (12) show that $(M_c - M_s)$ and $(D_x - D_y)$ vary linearly with $(V - H)$. Strong evidence that this specific case applied to section 4D throughout most of its loading history is shown by the linearity of the plot of observed $(M_c - M_s)$ vs. $(V - H)$ in Fig. 12 and by that of a similar plot of $(D_x - D_y)$ vs. $(V - H)$. It is indeed remarkable that the slotted concrete ring performed in such manner as to create the

same type of distribution for the horizontal as for the vertical load, particularly in view of the differences between the theoretical and actual ring.

Comparison of Slotted Concrete and Theoretical Ring

Necessarily, the actual conditions at section 4D included many departures from a theoretical ring, any one of which would tend to contribute further to the moment. Local variations in stiffness of the supporting ground were undoubtedly present. An obvious one was the possibility of the lignite layers acting as hard spots since both geologic evidence and laboratory tests on unfissured specimens showed the lignite stiffer than the surrounding hard clay. However, there was at least an equal possibility that the open joints initially present in the lignites and their subsequent enlargement by yielding toward the open bore would make the horizontal deformation characteristics of the lignite in situ comparable to that of the adjacent ground. Although not fully conclusive, the weight of evidence from the ground yield measured by the spearheads and the generally smooth shape changes, both of the steel ribs and of the concrete rings, suggest that the lignites approached the latter case of acting as one with the surrounding ground. Since grouting would tend to make the lignites stiffer, this was minimized except where it could not be avoided at crossings of the grout curtains.

The four slots decidedly acted as major discontinuities. While the original intention was to span the slots with a steel section having a load resistance comparable to that of the concrete lining, this objective was not realized in regard to bending resistance, although it was reasonably met with respect to thrust. Restudy showed the as-built moment of inertia (in units of inches^4 per lineal foot of tunnel) varied from a low of 26,000 at the slots to a maximum of 180,000 at regions of greater overbreak near the crown, with 96,000 being the average around the bore. This much lower stiffness at the slots tended to make them act as hinges.

The loads arched over to section 4D during adjacent mining would be expected to be inclined and some evidence of this was afforded by the tendency of the crown to roll away from the side where adjacent mining was underway. Since the thrusts and moments in section 4D were measured only on horizontal and vertical axes, they would not represent the maximum values created by an inclined load, at least in respect to finding any of the theoretical moment coefficients applicable.

Load on Slotted Concrete Section

Limitations of Ring Theory

The theoretical equations for moments and deflections of rings reasonably available in the technical literature (see Table 4) are necessarily for idealized cases and consider a relatively thin ring of homogeneous, isotropic, elastic material with a constant cross sectional moment of inertia. The deflections are those resulting from moment alone; the additional deflections from thrust are neglected. Many of the stress equations for reinforced concrete are to a considerable degree empirical from experimental studies with beams, so it is questionable if these equations are adequate for concrete rings. While other factors could be listed, these are sufficient to indicate the considerable limitations of available ring theory.

Probably even more significant were the complicated conditions of the actual case, with the ring possessing a variable moment of inertia, with the part-elastic part-plastic behavior of actual materials and with the horizontal load apparently dependent upon ring flexure and upon the vertical load to a greater degree than visualized by the theoretical solutions. Hence, in seeking a load distribution compatible with performance of the slotted test section, close agreement between theory and measurement was not expected and was not obtained.

Load Distribution

The general procedure was to determine by trial the type of load distribution best fitting the measured data. Theoretical ring coefficients for several basic load diagrams are shown in Table 4 as available from solutions based on the theory of elasticity.[8, 9, 10] More complex distributions were obtained by combining these basic diagrams. While several procedures were tried, final studies were concentrated on the following two.

The first procedure utilized the linear relationship found between (M_c - M_s) and (V - H). Fig. 14 shows the result where a horn shaped loading (distribution No. 29) gave a reasonable fit with the measured data for nearly the entire loading period up to August 1950. At this time adjacent mining began in tunnel 3 and there was a change in the load distribution as shown by the offset in the curve of Fig. 12.

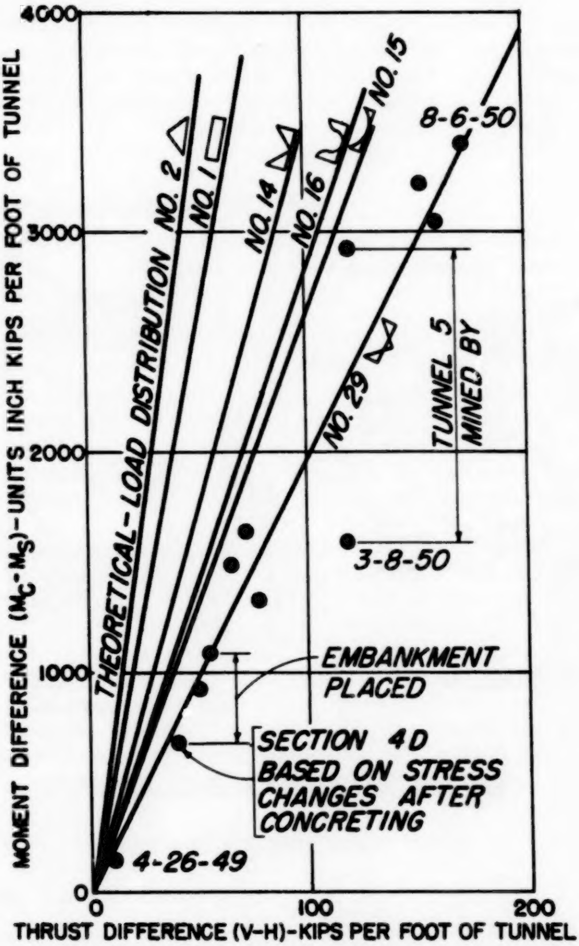
The second procedure involved assuming a type of load distribution, computing its moment and diameter changes and then comparing these with the measured data. To handle a number of load distributions, the study was conducted in terms of dimensionless ratios: H/V, M_c/M_s and D_x/D_y. Effect of a variable I was investigated for a few cases and found to be considerable; however, the solution was too complex for general application. This second procedure also indicated a horn load, with a very reasonable fit for the moment data and a poorer but fair fit for the diameter change data.

The conclusion from both procedures was a horn load combined with a small uniform load which is shown by Fig. 15 for data of September 1949 when placement of the embankment was nearly complete. While data from subsequent dates involving heavier loading did not fit equally well, the agreement was sufficient to indicate that a horn type of load distribution prevailed for most of the loading history of section 4D.

Effect of Slots

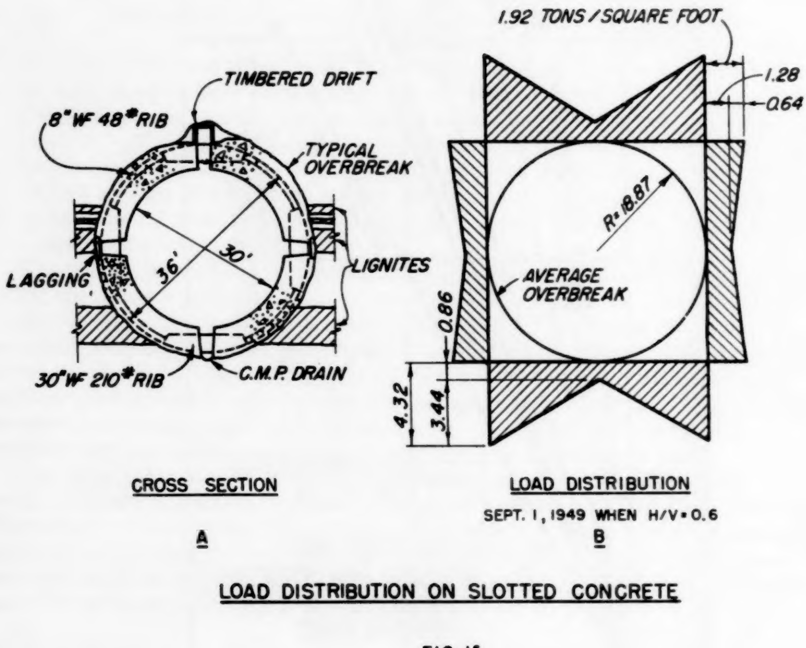
While it was hardly visualized in planning the test tunnel that the slots would affect the load distribution to the degree indicated by Fig. 15, reconsideration in the light of the arching and relative yield concepts showed that the details of the slot construction could permit significant yield, thus causing the slots to shed load. For convenience in later concreting of the top slot, the contractor was allowed to place a 3 x 3-foot drift at the crown. The

8. "The Structural Design of Flexible Pipe Culverts," by M. G. Spangler, Public Roads, Vol. 18, No. 12, Feb. 1938, Pub. by Bureau of Public Roads, Washington, D. C.
9. "Elastic Energy Theory," by J. A. Van Den Brock, John Wiley & Sons, New York, 2nd. Ed., 1946, P. 283.
10. "Determination of Lateral Passive Soil Pressure and Its Effect on Tunnel Stresses," by M. A. Drucker, Journal of Franklin Institute, May 1943.



MOMENT DIFFERENCE V.S. THRUST DIFFERENCE

FIG. 14



CROSS SECTION

A

LOAD DISTRIBUTION

SEPT. 1, 1949 WHEN H/V = 0.6

B

LOAD DISTRIBUTION ON SLOTTED CONCRETE

FIG. 15

increased width from overbreak and the relatively light timbering of this drift acted to permit more yield at the crown than at the other slots—see Fig. 15. At the invert a section of half-round corrugated metal pipe was placed as a drain while at both spring lines 10-inch channel lagging was placed on its side to span between the rib flanges and back-packed with sand.

All of these slot details would permit some yield relative to the adjacent concrete lining which would result in the load arching over to either side of the slot—a maximum at the crown and probably least at the spring lines. This yielding ability of the slots offered a confirmation of the horn load as a feasible load distribution and created doubt if the slotted concrete section was an acceptable model of the prototype tunnels.

Slotted Section as a Model of the Prototype

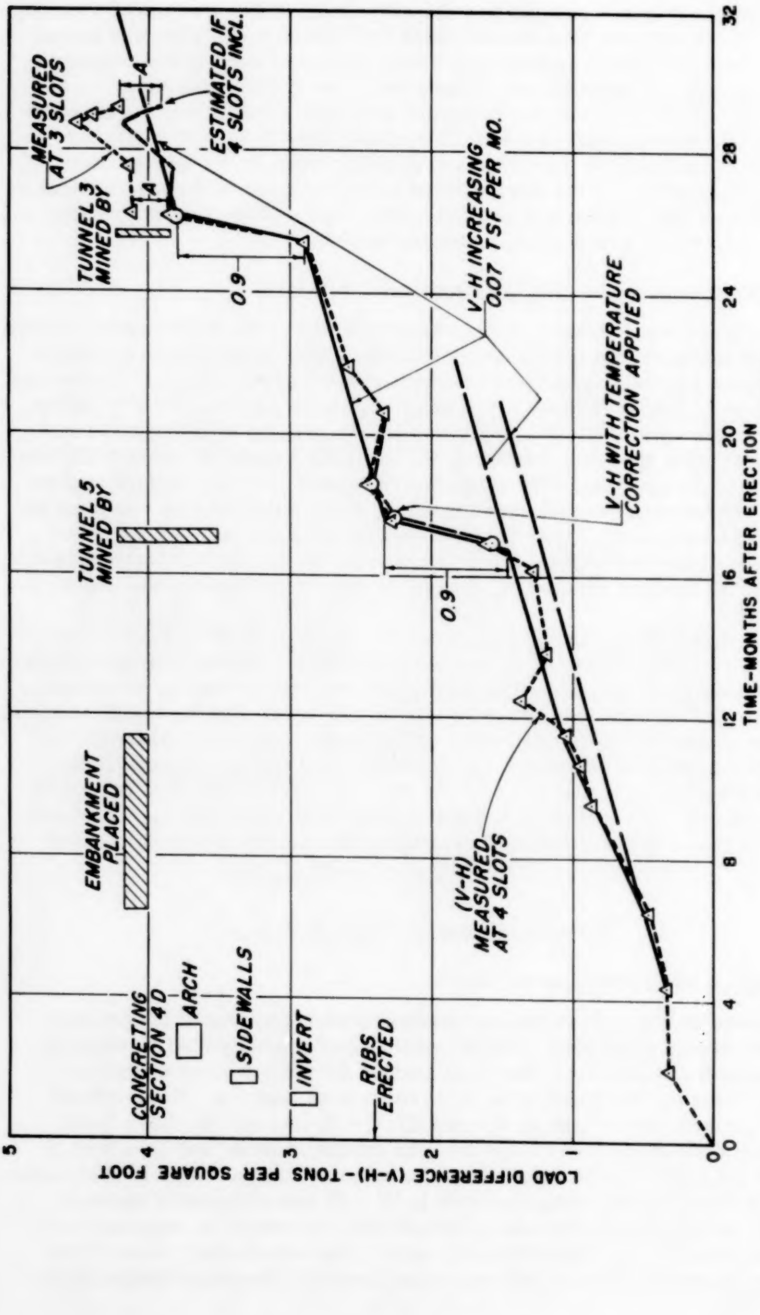
Since there were no slots in the concrete lining of the main tunnels, a horn load distribution would not be all applicable. Also the full circle prototype lining would be stiffer and hence undergo less deflection than the test section with the slots acting as hinges. As shown by Table 4 of Mr. Burke's paper, the deflection varied with the flexibility of the concrete lining—slotted test section 4D being the most flexible of the sections measured, section $4B_1$ the least due to its approximately 4-foot lining thickness, and prototype tunnels 2 and 5 representing an intermediate case. The slotted section could not be a true model of the prototype unless it were possible to correct for this difference in flexibility since the vertical load, the horizontal-vertical load ratio and the bending moment all depend in varying degrees on the lining flexibility.

The data did not permit quantitive evaluation of such correction for the moderately greater stiffness of the prototype lining. However, from a qualitative standpoint it appeared that the slotted test section was an acceptable model for the vertical load, but only fair for the horizontal load, and no model at all for either the load distribution or the bending moment. Although the unintended action of the slots as partial hinges and the unforeseen yielding permitted by the as-built details of the slot construction resulted in slotted section 4D being far less of a model than originally hoped for, nevertheless its performance was a great aid in clarifying the action of circular tunnel lining.

Action of Circular Tunnel Lining

Performance of Slotted Test Section

As shown by Fig. 11 the various loading events (addition of the embankment and mining of adjacent tunnels) each caused a pronounced increase in load, moment and diameter change at section 4D while a gradual increase took place during the intervals between these load changes. These effects are particularly well shown by the plot of (V - H) on Fig. 16. Here trend lines have been drawn to average the data and these show that the effect of adjacent mining in tunnel 3 was about the same as that in tunnel 5. They also show that the rate of gradual increase in (V - H) was essentially constant over the period of measurement, although this rate would be expected to decrease as the vertical load eventually approached its ultimate value. This increase between loading events was most likely due to consolidation of the



V-H vs TIME-SLOTTED CONCRETE SECTION

FIG.16

adjacent ground pillars under the loading of the single tunnel case from mining section 4D. Fig. 16 can also be considered a curve of V/2, since from Fig. 9

$$H = \frac{1}{2} V \qquad \text{so} \qquad V - H = \frac{1}{2} V = H$$

and Fig. 16 is also a curve of H.

The linear relation of the plot of (V - H) vs. (M_c - M_s) in Fig. 12 and also of (V - H) vs. (D_x - D_y) is strong evidence that both V and H had the same type of load distribution curve which was one of the more unexpected results of the measurements.

Effect of Stiffness

Perhaps the most consistent results from the various types of data was the tendency for the flexible rib support to shed load and for the stiff concrete lining to attract it. The values of moment were higher than anticipated and increased decidedly with stiffness as shown by Fig. 13. For the flexible ribs, H/V was approximately equal to one and the moment moderate but still controlling in selection of rib section. For the slotted concrete section H/V was approximately one-half and the moment high. All told, the stiffness of the tunnel appeared to be the principal factor controlling the vertical and the horizontal load and from these the moment. From equation (7) stiffness would appear to vary inversely with the diametric strain and thus to be some function of $\frac{EI}{R^3}$.

Ring Action

From the over-all investigation it appears that a circular tunnel ring can be visualized as carrying its activating load V as one increment of V which is balanced by the horizontal load H, and as a second increment which is supplied by the bending resistance of the ring, B_R. Where V and H have the same type of distribution, as was indicated for the Garrison tunnels, equation (11) can be written as

$$(V - H) = F\,(M) \qquad \text{or} \qquad V = H + F\,(M) \tag{15}$$

where the moment function may approach the bending resistance of the ring.

If the ring is very flexible, having only a negligible bending resistance, it readily deforms, creating a passive horizontal load

$$H = V \qquad \text{and as} \qquad V - H = 0 \qquad M = 0 = B_R$$

If the ring is very stiff with a large bending resistance, then the deformation is small, H is small, (V - H) is large and M is high. As a limiting case with H negligible

$$H = 0 \qquad V - H = V \qquad M = B_R$$

Thus, with some over-simplification, this concept can be represented by writing equation (15) in the form

$$V = H + B_R \tag{16}$$

This illustrates the automatic safety inherent in a circular tunnel ring which is capable of self adjustment to accommodate its load conditions. If the vertical load approaches or exceeds B_R, the ring deforms further, building up the passive load H which then permits the ring to carry an even greater load. Should the combined stress from moment and thrust reach the yield point, the bending resistance may be reduced but the ring is then more flexible so its deflection increases, further increasing H, with any reduction in B_R being offset by the increase in H. Thus a circular ring is capable of adjusting itself to carry the activating load V by a combination of stresses and deformations which very likely follow the principles of least work.

If the concrete lining is so stiff that its high bending resistance initially carries most of the vertical load with resulting high fiber stress, cracking of the lining will then make the ring more flexible so that it reaches equilibrium in carrying further increments of the vertical load by building up the passive load H. As a limiting case in a considerably cracked condition, the lining might approach a segmented arch as a series of hinged blocks, able to take high thrust but low moment and possessing considerable flexibility. This would be somewhat like the English practice of using precast concrete segments, in lieu of cast iron segments, with a thin sheet of compressible material in each circumferential joint to introduce some flexibility.(11) Such a segmented ring is entirely safe and actually able to carry a considerably higher load without over-stress than could the stiffer ring before cracking, provided the deformations are small and the ring maintains its integrity without buckling.

Performance of Prototype Lining

Some cracking has appeared in the concrete lining of the main tunnels, generally as fine hair cracks in a fairly consistent pattern of longitudinal cracks at the crown and invert and of transverse cracks which seldom extend below the arch. Cracking is normal for reinforced concrete under significant loading since when the stress in the reinforcing steel reaches even a moderate tension, the outer fiber of the few inches of concrete cover over the reinforcement is stressed beyond the tensile strength of the concrete. The longitudinal cracks are considered due to moment and the transverse cracks due to a combination of shrinkage, temperature, flexure from settlement and rebound, plus a direct tension from the spreading effect of the triangular load of the dam. Some evidence showed the cracking was accentuated by sharp temperature drops when winter winds blew through the tunnels. This exposure was more severe than operating conditions and could be minimized on future projects by use of temporary bulkheads. The cracks were mapped during final inspection in the spring of 1951; and it is significant that nearly all of the cracking was found in tunnels 2, 5 and 7, which were subjected to adjacent mining, and that practically no cracking was evident in tunnels 3 and 6 which were constructed between previously concreted tunnels. This confirms the

11. "Tunnel Lining, With Special Reference to a New Form of Reinforced Concrete Lining," by G. L. Groves, Jour. Inst. of Civil Engineers, London, March 1943.

relative yield method in indicating the multiple tunnel case as responsible for the most severe loading.

There is no question but the prototype tunnels are heavily loaded, particularly those subjected to adjacent mining. For such tunnels, load estimates have been made on the assumptions that: (1) V reached 100% overburden, (2) H and V had the same type of distribution and (3) cracking during the multiple tunnel loading decreased the stiffness of the prototype to approach that of the slotted test section and thus result in H/V = 1/2. This study resulted in a curve similar to Fig. 16 as modified to fit actual time of loading events and to reflect the increased overburden load at center of dam—one study being based on H/V = 1/2 and one on H/V less than this prior to cracking. For either study, available theory indicated large diameter changes of several inches while the actual diameter changes were only a small fraction of an inch, which illustrates the inadequacy of the ring theory. Hence, a more realistic appraisal is afforded by the smooth shape changes and the small diameter changes actually measured in the prototype (around 1/4 to 1/3 inches) which show that the tunnels are performing quite satisfactorily, fully preserving their integrity and utilizing the automatic safety of ring action in adjusting themselves to the loading conditions.

Diameter measurements were continued at a few locations on the concrete lining until the tunnels were flooded by diversion in the spring of 1953 and gave some evidence that the tunnels mined between previously concreted tunnels have begun to share the load. When the tunnels were successively unwatered in the spring of 1954, more limited measurements showed only minor diameter changes during the elapsed year, thus indicating that the tunnels are approaching equilibrium. From spot checks the cracking did not appear to have increased in the five power tunnels but had done so in smaller tunnels 6, 7 and 8. This might be explained from the greater ring stiffness of these smaller diameter tunnels as stiffness probably varies inversely with the cube of the diameter.

Application to Rib Design

Original Design

For contract advertising purposes an initial rib design was developed in the fall of 1948 when sections 4A and 4C of the test tunnel had been supported by ribs for about three months. These test sections were then carrying approximately 15% of the overburden load which was reasonably checked by the arching method based on the initial shear strength s_1 as indicated in Table 2. Computation by this same method indicated rib loading in the range of 30 - 60% overburden for tunnels 1 to 5 and 20 - 35% for the smaller tunnels 6, 7 and 8.[12] From this the ribs were designed principally for carrying thrust caused by 50% overburden for tunnels 1 to 5 and 40% for tunnels 6, 7 and 8. This resulted in a 3-foot spacing of 10-inch 49 to 72 pound ribs beneath the

12. It is interesting that this analysis by the arching method indicated the heaviest loading on those tunnels mined into an area already stressed by the load arched over during mining of the two adjacent tunnels. The actual condition was quite different as these tunnels mined between previously concreted tunnels experienced only a moderate initial load which was more correctly evaluated by the relative yield method.

center of dam and 8-inch 35 to 48 pound ribs at the end sections near toes of the dam—the ribs being proportioned for the variations in overburden load shown by Fig. 1. The contract was awarded on a unit price per pound with provision to vary the rib spacing up to 4 feet, which experience in the test tunnel had shown as about the safe maximum for supporting loose blocks.

Evolution of As-Built Design

Early results, first in tunnel 7 and then in tunnel 5, showed the measured thrust considerably smaller than estimated from the arching method and this led to development of the relative yield method as a more realistic approach. However, the bending moment was much higher than anticipated, or than could be accounted for by the small difference between the measured vertical and horizontal thrusts, and was responsible for a very high percentage of the total combined stress. With the moment thus apparently dependent on other than the over-all loading, the empirical relationship of Fig. 13 between moment and rib stiffness was developed as rapidly as data became available from the various rib measuring sections. For revising the rib design, the moment criterion was taken from this job experience curve as twice the average moment measured in the pertinent section but not less than the maximum measured on any individual rib. The reason for considering an individual maximum was to guard against the crippling of one rib starting a progressive failure in the period of temporary support when the ribs act individually prior to their encasement in concrete. Fig. 13 shows these moment design values which were combined with thrust estimated by the relative yield method and used for redesigning the ribs.

The as-built design evolved in several stages as further data became available and generally resulted in a 4-foot rib spacing except in approximately the first half of tunnels 5 and 7 and in local regions where blocky ground required closer support from a safety standpoint. It was necessary at several times to use ribs on hand or then being fabricated so that the final construction was occasionally heavier than needed. However, numerous measurements, plus considerations of cross-bending previously mentioned, indicated that the outer fiber stress probably approached the yield point on many ribs, so that the as-built design came close to the practical limit of economy. The saving in rib steel amounted to over 4,500,000 pounds or about $750,000 below the original design.

Suggested Future Procedure

From this experience, the following procedure deserves consideration in reasonably similar work.

1. Estimate maximum permissible rib spacing from the safety standpoint of preventing falls of loose blocks, as judged from character of the formation, experience with a test drift, etc.
2. Plan so that the ribs are required to furnish support only for the single tunnel case for a moderate period before being encased in concrete.
3. Estimate the vertical load by the method of relative yield and estimate the horizontal load on the basis that the more flexible the rib ring, the more nearly will the horizontal approach the vertical load.
4. Estimate the maximum bending moment, considering that it increases with rib stiffness and with wider blocking spacing.

5. Design the ribs for a working stress reasonably below the yield point and provide for an opportunity to vary the rib spacing (and under some conditions possibly also the cross-section).

6. Install occasional groups of adjacent ribs with strain gage lines and measure the actual stresses. As job experience and the pattern of thrust and moment become established, modify the rib design for balance of the work.

CONCLUSIONS

As with most investigations some of the results were reasonably definite, some inconclusive and some unanticipated. The stress pattern in the steel ribs was adequately established, the major contribution somewhat unexpectedly being due to bending moment; and use of this information in redesigning the ribs resulted in a considerable saving in steel amounting to around three-quarters of a million dollars. Although less tangible, a much larger saving is believed to have accrued from the lower bids resulting from the opportunity the test tunnel afforded bidders to appraise the work with much less uncertainty than usual in tunneling.

Comparison of the concrete lining with the slotted test section was handicapped by the tendency of the slots to act as partial hinges. Thus the adequacy of the prototype full-circle lining was more realistically shown by its demonstrated ability to automatically adjust itself to the load conditions according to the principles of ring action. However, the slotted test section was very helpful in clarifying the action of tunnel lining which may aid in reducing some of the present inadequacies of ring theory. The over-all results of the investigation strongly indicated that with some over-simplification a circular tunnel lining can be considered as carrying part of the vertical load by its bending resistance and carrying the balance by yielding to build up the horizontal or passive load.

The effect of tunnel stiffness was very marked, both on the vertical load attracted and on the resulting horizontal load and bending moment. Where the tunnel was stiffer than the adjacent ground it attracted load, where less stiff it shed load, thus confirming Terzaghi's(13) comparison of rigid and flexible tunnels. For a single tunnel case (either one tunnel or a series of tunnels spread sufficiently to avoid inter-effects) increased application of this principle should result in significant savings by utilizing more flexible linings and relying on the ground to carry the major part of the overburden load.

The problem is more complex for the multiple tunnel case where the tunnels are sufficiently close that they affect each other. Under some conditions it should be possible to mine all tunnels and then allow the ground pillars to fully consolidate under their increased load before placing the permanent lining. Thus with no further deformation of the ground pillars, there would be no increase in load on the tunnel, which would permit a relatively light lining. This procedure was investigated but rejected for the Garrison tunnels since it would overload the ground pillars unless the construction schedule were unduly prolonged to allow time for the pillars to gain strength by consolidation before permitting any adjacent mining.

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13. "Liner Plate Tunnels On the Chicago (Ill.) Subway," by K. Terzaghi, Trans. ASCE, Vol. 108, 1943, p. 970.

Thus for the relatively closely spaced Garrison tunnels, it was considered necessary to design the concrete lining for a thrust of 100% overburden and the data showed that this load was essentially reached, at least on those tunnels subjected to adjacent mining. While reinforced concrete has many advantages, the nature of the material is such that proportioning to carry such a high thrust inevitably leads to a relatively thick section of considerable stiffness with its accompanying disadvantage of a tendency to carry much of the load by its bending resistance. Some study was given to a scheme of carrying the permanent loads by a more flexible section utilizing heavier rib steel and lighter concrete; but this seemed to offer no cost advantage and appeared likely to increase the construction problems. For tunnels required to carry high thrust, it seems likely that the ideal tunnel lining has yet to be developed, or at least to receive any wide acceptance.

ACKNOWLEDGEMENTS

During the course of this tunnel investigation Brig. General W. W. Wanamaker and Major General J. S. Seybold served in turn as District Engineer of the Garrison District, Corps of Engineers, with R. H. Hayes as Chief, Engineering Division; R. D. Field as Chief, Design Branch; and H. F. Michel as Head, Structural and Civil Section. The investigation was conducted under the direction of K. S. Lane as Chief, Soils and Geology Branch with the work divided between H. H. Burke as Head, Field Test Section handling the measurements and particularly assisted by K. T. Lord; and C. K. Smith, as Head, Soils Design Section handling analysis of the results and particularly assisted by R. E. Torp-Smith. Advice in planning the test tunnel and interpretation of the results was obtained from a Board of Advisors consisting of G. E. Bertram, R. B. Peck, R. R. Philippe, W. J. Turnbull, and the late D. W. Taylor. With the exception of Messrs. Bertram and Lord, all are members of the ASCE.

APPENDIX A

Vertical Load by Arching Theory

The idea of ground arching around a yielding support has been developed by Cain,[14] Marston,[15] Terzaghi[16] and others. Fig. 6 shows this concept applied to a tunnel beneath a considerable depth of overburden where part of the overburden load W is transferred by shears S on vertical planes at each side of the tunnel. These vertical planes have been taken one diameter apart in view of evidence from lignite mines in the Fort Union where collapse of a drift usually produces a depression at the surface which is not much wider than the drift. For judging the probable distribution of the force S, the theoretical solution for stress around a yielding hole in a uniform stress field appeared reasonably applicable. Such is plotted on the left side of Fig. 17a as found from the theory of elasticity and verified by photoelastic experiments[17] and shows the stress a maximum at the crown, decreasing to a comparatively small value about 2 diameters above the crown.

Assuming a constant shear modulus, making the deformation directly proportional to shear stress, the deformation diagram would then be of the same shape as the diagram of shear stress. Fig. 17b shows the ground movements measured as settlement of telescopic sections of the vent shaft located above section 4C of the test tunnel. The similarity between the diagram of measured movements and that of the theoretical shear stress distribution is good evidence of the applicability of the latter, although it was derived for a case of uniform initial stress in a vertical direction only. Since the objective is a workable means of estimating the vertical tunnel load, this analysis neglects any initial horizontal stress field—the variation of the properties of the ground and the dependence of the horizontal load on the magnitude of the vertical being of such complexity that such refinement seems unjustified.

For computing purposes the theoretical distribution curves are simplified to triangles as shown on the right side of Fig. 17a. The area of the triangle d-e-f represents the total shear on the plane d-e and equals the vertical stress increase on the plane d-j, represented by the area of the triangle d-j-k. It is assumed that the shear stress f-g at the crown reaches the available shear strength s, which is represented by the Coulomb equation

$$s = c + K \sigma \tan \phi \qquad (17)$$

where the no-load shear strength c is taken as 0.7 t.s.f. and the friction angle ϕ as 20°—these being the usual design values for Fort Union which were derived principally from studying the shear strength required to maintain

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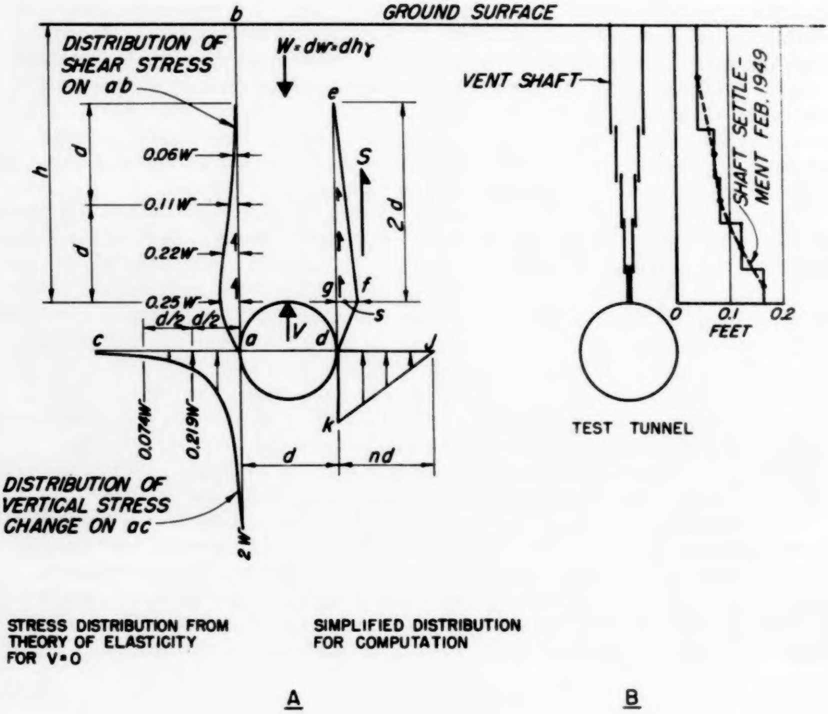
14. "Earth Pressure, Walls & Bins," by W. Cain, John Wiley & Sons, 1916, p. 208.
15. "The Theory of Loads on Pipes in Ditches and Tests of Cement and Clay Drain Tile and Sewer Pipe," by A. Marston and A. O. Anderson, A.S.T.M. Proc., 1913, P. 13 and 303-312.
16. "Theoretical Soil Mechanics," by K. Terzaghi, John Wiley & Sons, 1943, p. 69.
17. "Photoelasticity," by M. Frocht, John Wiley & Sons, 1941, Vol. I, p. 225.



ARCHING THEORY

FIG. 17

existing slopes as described by Smith and Redlinger.[4] The stress σ is the vertical effective or inter-grandular stress and K is the ratio of horizontal to vertical stress. The value of K is uncertain and here is assumed as equal to one, although it could be greater in event of high horizontal stress remaining from the load of past overburden as mentioned previously in the discussion of the horizontal-vertical load ratio.

From Fig. 17a the tunnel load is then

$$V = W - 2\,S = dw - 2.5\,d\,s \tag{17}$$

Since W increases with the depth h faster than does the shear strength s, this equation shows, as expected, that V increases with h and d. The biggest uncertainty is the evaluation of the factors controlling the shear strength, principally the stress ratio K and the increase in the effective stress σ due to the progress of consolidation as discussed in the portion of the paper covering the arching method. The factor n in Fig. 6 represents the distance arched and is pertinent only to a multiple tunnel case where the second tunnel is mined into a region of ground previously stressed by the load arched over during the mining of the first tunnel. For tunnels at depths less than about 3 diameters it is apparent from Fig. 17a that the distribution of S would need to be modified.

APPENDIX B

Vertical Load by Relative Yield Theory

This method was developed initially by C. K. Smith[5] by modifying the method of arching to allow for the effect of varying degrees of rigidity supplied by the tunnel supports. Fig. 7 shows the basic concept with a series of elastic blocks at the plane of the tunnels somewhat like the blocks considered by Cain[18] in studies of culvert loading. These blocks deform according to Hooke's law in direct proportion to their loading but have different deformation moduli: E_t for the rib supported tunnel, E_s for the ground and E_c for the concrete lined tunnel, such that

$$E_t < E_s < E_c$$

The lower boundary at the invert is considered a plane of no movement or at least of uniform settlement. Ground above the tunnels is considered as a series of blocks whose height h is so much greater than their width d that each block is rigid within itself. Loads are transferred between the blocks by shear on their boundaries, as in the arching concept, although in this case the distribution of this shear stress is not pertinent.

With these simplifying assumptions the relations for the tunnel loads can be obtained by simple statics. The mathematics of the derivation have been

4. Prior reference.
5. Prior reference.
18. "Earth Pressure Experiments on Culvert Pipe," by W. Cain, G. M. Braune and H. F. Janda, Appendix I by W. Cain, Public Roads, Nov. 1929, Pub. by Bureau of Public Roads, Washington, D. C.

intentionally kept simple, using large finite blocks instead of infinitesimals, since the numerous uncertainties in properties of both ground and tunnel supports do not warrant refinements. The basic symbols are defined by Fig. 7.

Single Tunnel Case

Consider a single tunnel, Fig. 18, mined with rib support sufficiently flexible that the modulus of the tunnel block E_t is less than that of the ground E_s, so that

$$E_t = bE_s$$

In the tunnel block

$$\text{Strain} = \frac{\text{Stress}}{E_t} = \frac{y}{d} = \frac{\frac{W - 2T}{d}}{bE_s} \quad \text{or} \quad y = \frac{W - 2T}{bE_s} \tag{18}$$

In the adjacent soil block, the transferred load T causes an average deflection y/2

$$\frac{\frac{y}{2}}{d} = \frac{\frac{T}{nd}}{E_s} \quad \text{or} \quad y = \frac{2T}{nE_s} \tag{19}$$

Since the deflection must be the same at the boundary between blocks, equating (18) and (19) results in

$$T = \frac{n\,W}{2(b + n)} \tag{20}$$

Whence the tunnel load is

$$V = W - 2T = W\left[1 - \frac{n}{b + n}\right] \tag{21}$$

Surface Loading Case

Consider a unit load p added at the surface which produces a general deflection y_2, Fig. 19.

$$\frac{y_2}{d} = \frac{p}{E_s} \quad \text{or} \quad y_2 = \frac{pd}{E_s} \tag{22}$$

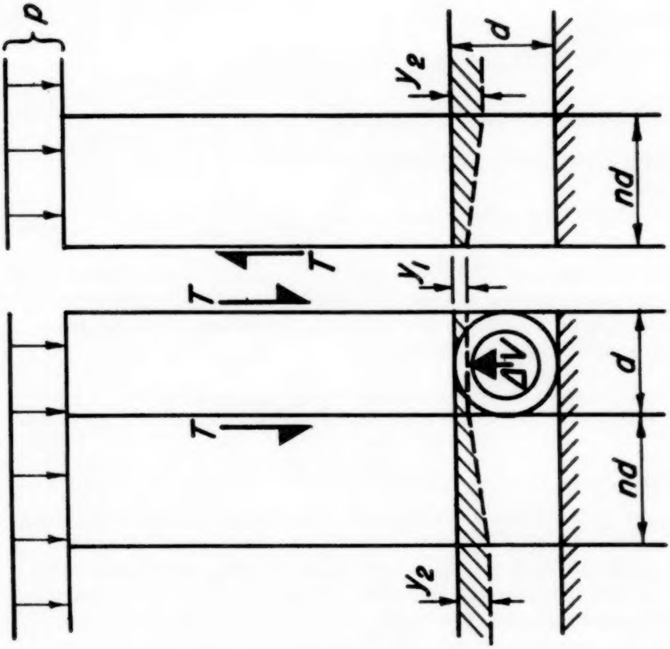
Let the tunnel be stiffer than the soil, so

$$y_1 < y_2 \quad \text{and} \quad E_c = cE_s$$

Then in the tunnel block

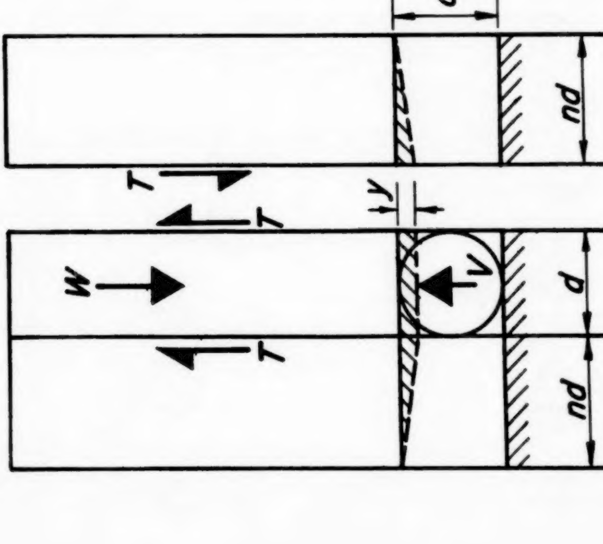
$$\frac{y_1}{d} = \frac{\frac{pd + 2T}{d}}{cE_s} \quad \text{or} \quad y_1 = \frac{pd + 2T}{cE_s} \tag{23}$$

and in the block adjacent to the tunnel where T causes the difference in deflection $(y_2 - y_1)$ or an average of half this



SURFACE LOADING CASE

FIG. 19



SINGLE TUNNEL CASE

FIG. 18

$$\frac{\frac{y_2 - y_1}{2}}{d} = \frac{\frac{T}{nd}}{E_s} \quad \text{or} \quad y_2 - y_1 = \frac{2\,T}{nE_s} \tag{24}$$

Substituting (22) and (23) for y_2 and y_1 in (24) results in

$$2\,T = pd\,\frac{n\,(c - 1)}{c + n} \tag{25}$$

Whence the load increase on the concreted tunnel is

$$\Delta V = pd + 2\,T = pd\left[1 + \frac{n(c - 1}{c + n}\right] \tag{26}$$

By substituting b for c, equation (26) then covers the load increase on a tunnel with flexible rib support; however, as $b<1$, $(b - 1)$ is negative, whence the rib support equation can be written in the following more usable form, the shears T being reversed from those in Fig. 19.

$$\Delta V(\text{ribs}) = pd - 2\,T = pd\left[1 - \frac{n(1 - b)}{b + n}\right] \tag{26a}$$

Multiple Tunnel Case

Consider an array of 3 tunnels as in Fig. 20. The relatively stiff concrete lining has been placed in tunnels 2 and 4. Tunnel 3 is now mined with its more flexible rib support which causes a load increase ΔP_1 on the two adjacent pillars and a load increase ΔV on tunnels 2 and 4. To keep the problem within the realm of simple statics, the depth h is assumed to be sufficient that the soil above the tunnels acts as a rigid beam resulting in a constant deflection y over the 3 tunnels. A portion of the overburden load W above tunnel 3 is then transferred to each side by the shears T_1, T_2 and T_3 as indicated. Relations between the deformation moduli of pillar and tunnel blocks are

$$E_c > E_s > E_t \qquad E_c = cE_s \qquad E_t = bE_s$$

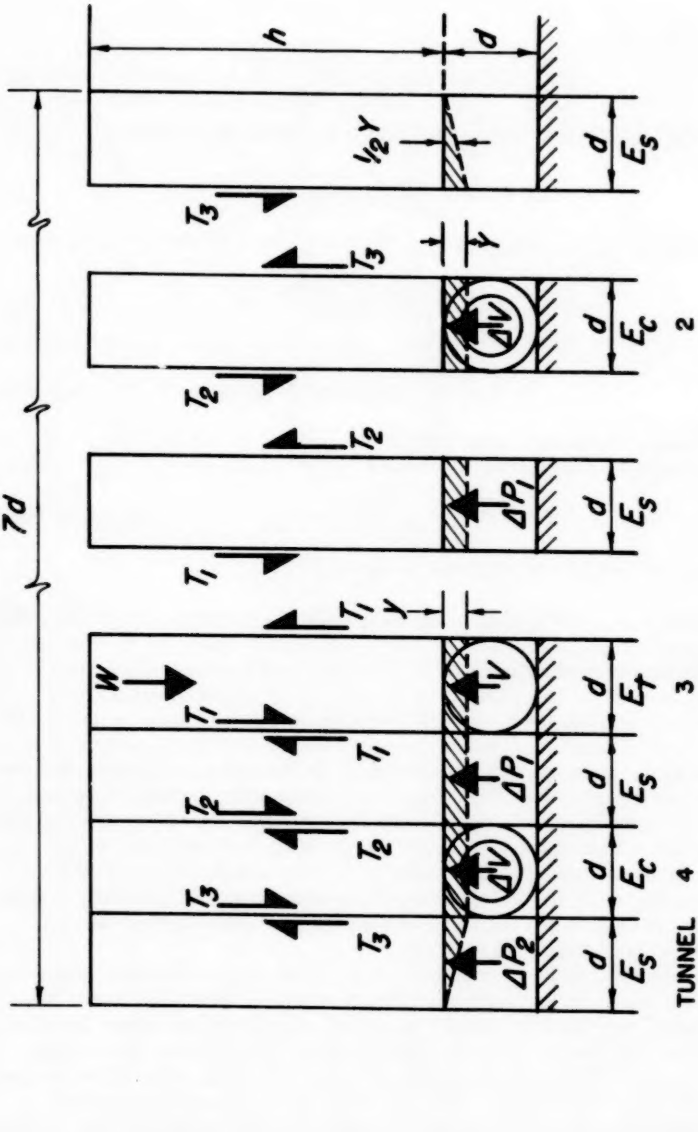
The deflection equations in the 4 different types of blocks are then

$$\frac{y}{d} = \frac{\frac{W - 2\,T_1}{d}}{bE_s} \quad \text{or} \quad y = \frac{W - 2\,T_1}{bE_s} \tag{27}$$

$$\frac{y}{d} = \frac{\frac{T_1 - T_2}{d}}{E_s} \quad \text{or} \quad y = \frac{T_1 - T_2}{E_s} \tag{28}$$

$$\frac{y}{d} = \frac{\frac{T_2 - T_3}{d}}{cE_s} \quad \text{or} \quad y = \frac{T_2 - T_3}{cE_s} \tag{29}$$

$$\frac{\frac{y}{2}}{d} = \frac{\frac{T_3}{d}}{E_s} \quad \text{or} \quad y = \frac{2T_3}{E_s} \tag{30}$$



MULTIPLE TUNNEL CASE

FIG.20

Equating (27) and (28)

$$T_1 = \frac{W + bT_2}{b + 2} \tag{31}$$

Equating (29) and (30)

$$T_3 = \frac{T_2}{(2c + 1)} \tag{32}$$

Substituting (31) in (28) and (32) in (29) and then equating, results in

$$T_2 = \frac{W}{2}\left[\frac{(2c + 1)}{(b + 2c + 3)}\right] \tag{33}$$

Substituting (33) in (31) and then in (32)

$$T_1 = \frac{W}{2}\left[\frac{(2c + 3)}{(b + 2c + 3)}\right] \tag{34}$$

$$T_3 = \frac{W}{2}\left[\frac{1}{(b + 2c + 3)}\right] \tag{35}$$

Whence the tunnel and pillar loads are:
Load on the temporary supports of tunnel 3

$$V = W - 2\,T_1 = W\,\frac{b}{(b + 2c + 3)} \tag{36}$$

Load increase on the pillars adjacent to tunnel 3

$$\Delta P_1 = T_1 - T_2 = W\frac{1}{(b + 2c + 3)} \tag{37}$$

Load increase on the concrete lining of tunnels 2 and 4

$$\Delta V = T_2 - T_3 = W\,\frac{c}{(b + 2c + 3)} \tag{38}$$

Numerous other assumptions are possible, as for example the average deflection of the concreted tunnel could be assumed as y/2 which would make the length of the soil beam 5d. This would be somewhat more compatible with the observed deflection of the concreted tunnel being less than that of the adjacent tunnel being mined, plus the tendency for the top of the concreted tunnel to roll away from the side where adjacent mining was underway. This change would increase V and decrease ΔV which again would be in slightly better agreement with the observed data.

Where the tunnels are located at a shallow depth, so that the soil beam above cannot be assumed as rigid, then it has appeared practical to derive equations considering the bending of this beam. Similarly the lower boundary or plane or zero movement could be taken at some depth below the tunnels. The work of Hetenyi[19] should be considered for solutions where it is necessary to consider the behavior of a flexible beam on an elastic foundation. These refinements would result in more complex equations and did not appear justified for the Garrison tunnels since the above simpler solutions gave results entirely adequate for practical usage.

19. "Beams on Elastic Foundations," by Hetenyi, Univ. of Michigan Studies, Scientific Series XVI, p. 97.

APPENDIX C

Theoretical Modulus of Rib Support

If actual measurements are lacking, the deformation modulus E_t for a rib supported tunnel can be estimated from the following equations for use as a first approximation in the relative yield method. For a case with a uniformly distributed load p, equations for the vertical diameter change D_y and the horizontal D_x of a circular ring are given in Table 4.

$$D_y = \frac{pR^4}{6\ EI} \tag{39}$$

Summing the effects of a vertical load V and a horizontal load H

$$D_y = \frac{V}{2R}\ \frac{R^4}{6\ EI} - \frac{H}{2R}\ \frac{R^4}{6\ EI} = \frac{R^3\ (V - H)}{12\ EI} \tag{40}$$

By definition the tunnel modulus $E_t = \frac{\text{stress}}{\text{strain}}$, where the stress is $\frac{V}{2RS}$ and the strain $\frac{D_y}{2R}$, S being the distance between ribs.

Hence

$$E_t = \frac{\frac{V}{2RS}}{\frac{R^3}{12\ EI}\ \frac{(V - H)}{2R}} \tag{41}$$

which reduces to

$$E_t = \frac{12\ EI}{R^3\ S(1 - \frac{H}{V})} \tag{42}$$

All of the values of equation (42) are either known or can be computed except for the H/V ratio which must be estimated. Furthermore, consideration should be given to the effect of bolted joints causing the actual rib to be somewhat more flexible and hence to have a lower value of E_t than might be obtained with a welded ring. In this connection an advance test was conducted on one ring, loading it with several cables at various diametric locations, with the general result that the more irregular the loading the greater did the bolted ring depart from the theoretical, while with more uniform loading it tended to approach but not equal the performance for a theoretical ring.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

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NOVEMBER: 1095(ST6), 1097(ST6), 1098(ST6), 1099(ST6), 1100(ST6), 1101(ST6), 1102(IR3), 1103 (IR3), 1104(IR3), 1105(IR3), 1106(ST6), 1107(ST6), 1108(ST6), 1109(AT3), 1110(AT3)[c], 1111(IR3)[c]; 1112(ST6)[c].

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127 (SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

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JANUARY: 1136(CP1), 1137(CP1), 1138(EM1), 1139(EM1), 1140(EM1), 1141(EM1), 1142(SM1), 1143(SM1), 1144(SM1), 1145(SM1), 1146(ST1), 1147(ST1), 1148(ST1), 1149(ST1), 1150(ST1), 1151(ST1), 1152(CP1)[c], 1153(HW1), 1154(EM1)[c], 1155(SM1)[c], 1156(ST1)[c], 1157(EM1), 1158 (EM1), 1159(SM1), 1160(SM1), 1161(SM1).

FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

MARCH: 1186(ST2), 1187(ST2), 1188(ST2), 1189(ST2), 1190(ST2), 1191(ST2), 1192(ST2)[c], 1193 (PL1), 1194(PL1), 1195(PL1).

APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203 (SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218 (SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226 (WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(HY2)[c].

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238(WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW2), 1245(HW2), 1246 (HW2), 1247(HW2), 1248(WW2), 1249(HW2), 1250(HW2), 1251(WW2), 1252(WW2), 1253(IR1), 1254(ST3), 1255(ST3), 1256(HW2), 1257(IR1)[c], 1258(HW2)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267 (PO3), 1268(PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275 (SA3), 1276(SA3), 1277(HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283 (HY3)[c], 1284(PO3), 1285(PO3), 1286(PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297(HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303 (ST4), 1304(ST4) 1305(SU1), 1306(SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312 (ST4), 1313(ST4), 1314(ST4), 1315(ST4), 1316(ST4), 1317(ST4), 1318 (ST4), 1319(SM3)[c], 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324(AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338(SA4), 1339(CO1) 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)[c], 1347(BD1), 1348(HY4)[c], 1349(SA4)[c], 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360(IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369(WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c], 1378(HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395(HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404(HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413(EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417 (HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)[c], 1431(ST6), 1432 (ST6), 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)[c], 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447 (SU2), 1448(SU2)[c].

c. Discussion of several papers, grouped by Divisions.